Technical Report REMR-CS-42
February 1994

US Army Corps
of Engineers
Waterways Experiment
Station

Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Overlays on Horizontal Concrete Surfaces: Case Histories

by Roy L. Campbell, Sr.
Structures Laboratory

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Prepared for Headquarters, U.S. Army Corps of Engineers

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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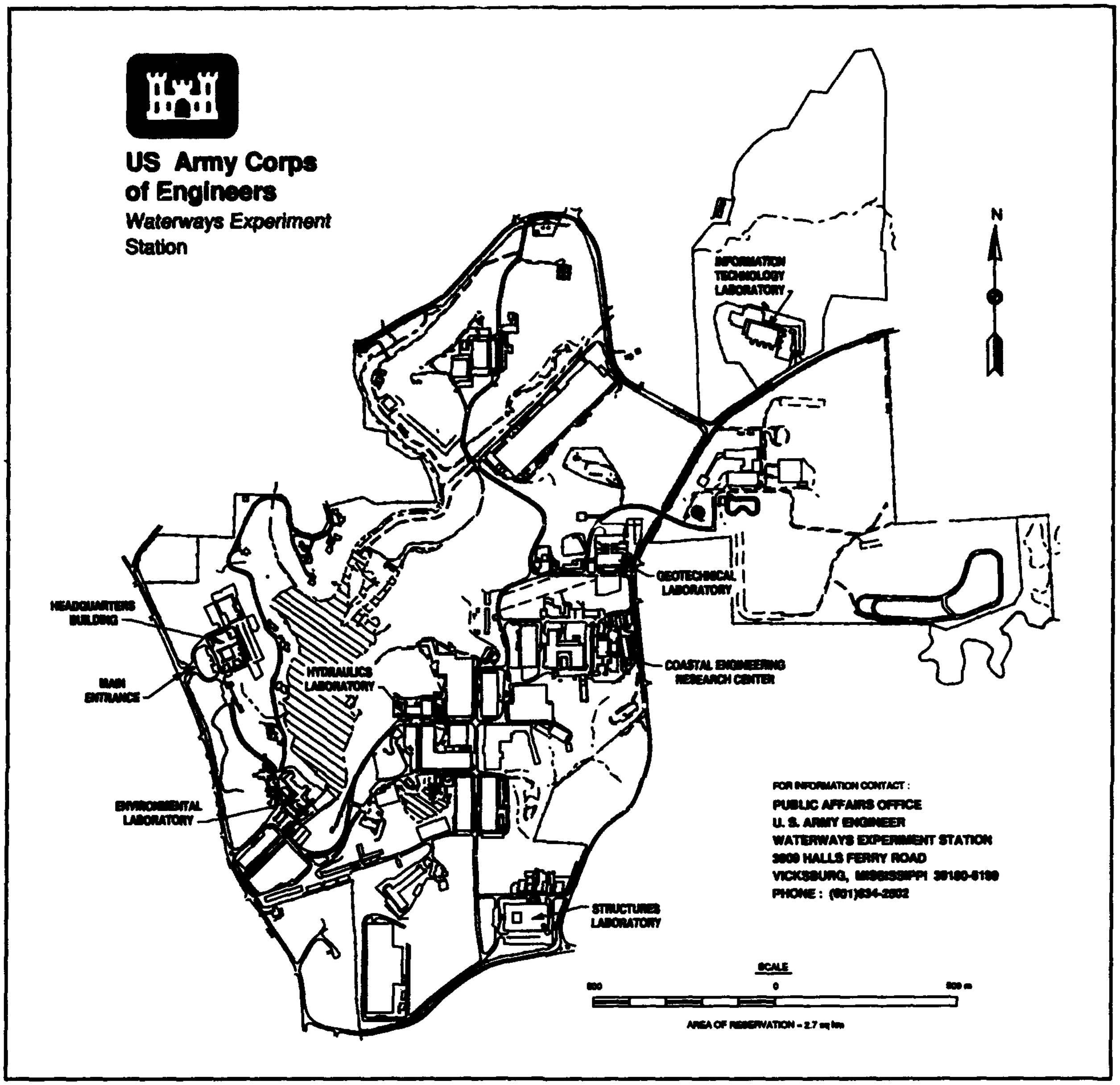

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Contents

Preface

The work described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32637, "Evaluation of Existing Repair Materials and Methods," for which Mr. James E. McDonald, Structures Laboratory (SL), U.S. Army Engineer Waterways Experiment Station (WES), is the Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program.

The REMR Technical Monitor is Dr. Tony C. Liu, HQUSACE. Mr. William N. Rushing (CERD-C) is the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Mr. James E. Crews and Dr. Liu (CECW-EG) serve as the REMR Overview Committee. Mr. William F. McCleese, WES, is the REMR Program Manager. Mr. McDonald is the Problem Area Leader for Concrete and Steel Structures. This report was prepared by Mr. Roy L. Campbell, Sr., Concrete Technology Division (CTD), SL, under the general supervision of Dr. Liu, Acting Chief, CTD; and Messrs. Bryant Mather and James T. Ballard, Director and Assistant Director, SL, respectively.

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At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI (Metric) Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
degrees (angle)	0.01745329	radians
Degrees Fahrenheit	5/9	Degrees Celsius or kelvins[1]
feet	0.3048	metres
fluid ounces	0.00002956353	millilitres
gallons (US liquid)	3.785412	litres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
ounces (mass)	0.02834952	kilograms
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic yard	0.5932764	kilograms per cubic metre
pounds (mass) per square yard	0.5424919	kilograms per square metre
square feet	0.09290304	square metres
square inches	6.452	square centimetres
square miles	2,589,998	square metres
square yards	0.8361274	square metres

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9) (F-32). To obtain kelvin (K) readings, use K = (5/9) (F-32) + 273.15.

1 Introduction

Background

Bonded overlays have been used for decades as a means to rehabilitate horizontal concrete surfaces that have deteriorated. The earlier overlays were conventional concrete mixtures that were solely portland cement based. Over the years, other materials such as air-entraining, water-reducing, and polymeric admixtures, fly ash, silica fume, and fibers were employed in an effort to improve the durability of the overlay and reduce the penetration of chlorides into the subjacent concrete. The resulting overlays included low-slump, fly-ash, silica-fume, polymer-modified, polymer, and fiber-reinforced concretes. For some nonstructural repairs, unbonded overlays have been employed in an effort to reduce reflective cracking and cracking resulting from the restrained contraction of the concrete overlay by the subjacent concrete. Overall, the performance and cost effectiveness of these various overlays are varied and unclear.

Purpose

The objective of this study was to document the current practices for overlaying horizontal concrete surfaces as the first phase in the development of performance criteria for concrete overlays.

Scope

Information was collected through (a) review of literature documenting the design, construction, and performance of the resurfacing repairs, (b) visits to project sites, and (c) discussions with personnel knowledgeable about the repair. Although the information obtained varied and was sometimes limited, an attempt was made to provide the following basic information for each repair: (a) a project description, (b) the cause and extent of the damage, (c) a description of the repair materials and procedures, (d) the cost of the repair, and (e) the performance to date of the repair. Based on a review and analysis of the information obtained, recommendations for future resurfacing were developed and areas which could benefit from research were identified.

For this study, an overlay was considered to be a conventional concrete overlay, if the sole binder for the concrete mixture was portland cement. An overlay was considered to be a low-slump concrete overlay and not a conventional concrete overlay if a) the sole binder for the concrete mixture was portland cement, b) the cement content was 800 lb/cu yd (475 kg/cu m) or greater, and c) the water-cement ratio was not greater than 0.4. A fly-ash concrete overlay was one in which fly ash was included in the concrete mixture; a silica-fume concrete overlay, one in which silica fume was included; polymer-modified concrete overlay, one in which a polymer admixture had been included; and fiber-reinforced concrete overlay, one in which either plastic or steel fibers were included. An unbonded overlay was one in which a bond breaker overlay was placed atop the existing concrete followed by the placement of the concrete overlay. The purpose of the bond breaker was to eliminate stress and subsequent cracking in the overlay that would result from the restraint of the existing concrete.

2 Conventional Concrete Overlays

Black Rock Lock

Background

Black Rock Lock is located in the U.S. Army Engineer District, Buffalo, on the Black Rock Channel and Tonawanda Harbor at Buffalo, NY. The project was completed in 1914 and includes a 625-ft (19-m) long by 68-ft (20.7-m) wide navigation lock. The concrete used to construct the lock was not air-entrained. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated.

Repair

The tops of the east lock wall in 1982 and the west lock wall in 1988 were resurfaced using conventional concrete containing welded-wire fabric (4 in. by 4 in. (102 mm by 102 mm)). New armor plating and reinforcement were provided along chamber face edge of overlay (Figure 1). The concrete specifications required a 37.5-mm (1-1/2-in.) nominal maximum size aggregate, a maximum water-cement ratio by mass of 0.44, an air content of 4.0 to 7.0 percent, a 1-in. to 4-in. (25-mm to 102-mm) slump, and a 28-day compressive strength of 4,000 psi (27.6 MPa).

Impact hammers were used to remove a minimum of 4 in. (102 mm) from surfaces. The impact hammers exerted considerable impact energy on the lock walls. Some concrete was difficult to remove, while drummy pockets were sometimes encountered. The concrete surfaces were specified to be cleaned by either wet sandblasting or water-jet blasting (minimum of 3,000-psi (20.7-MPa)). An epoxy bond coat was applied prior to concrete placement. Concrete was cured for a minimum of 7 days.

The total cost for the 1982 east wall repair was $600,000 and for the 1988 west wall repair, $700,000.

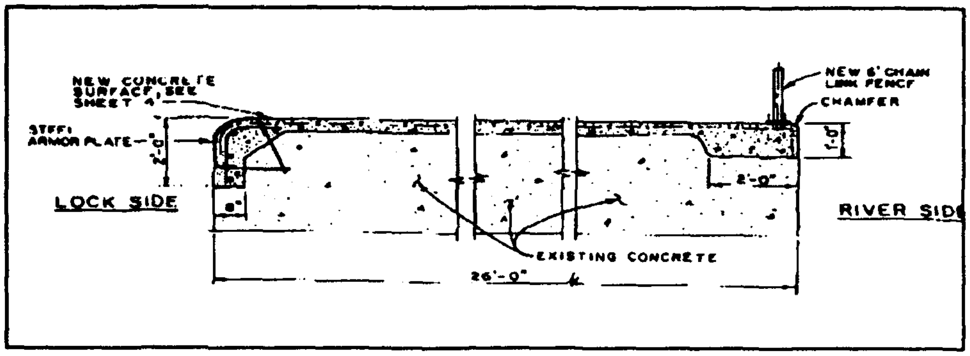

Figure 1. Typical overlay section, Black Rock Lock, 1988

Performance

Cracking was observed in the overlay that follows cracks observed in original surfaces.

Clearwater Dam Stilling Basin

Background

The Clearwater Dam is located in the U.S. Army Engineer District, Little Rock, on the Black River 5 miles southwest of Piedmont, MO. The dam is a rolled earthfill embankment with a concrete outlet works and an uncontrolled emergency spillway. The earth embankment was constructed with a single-stage diversion plan. The river was diverted through the previously completed outlet works by an upstream earth cofferdam, while the water downstream was retained by another lower earthen cofferdam. Construction of the outlet works was completed in 1942. No cofferdams nor river diversion were required for construction of the outlet works. Construction of the spillway and embankment was terminated in 1942 due to war conditions. Construction resumed in 1946, and the embankment and spillway were completed in 1948. The spillway weir was completed in 1951.

Divers equipped with an underwater video camera inspected the stilling basin in 1982 and found severe abrasion erosion damage in the stilling basin floor and baffle blocks. The area of deepest depression in the stilling basin floor was roughly "O" shaped and measured about 30 ft (9.1 m) long and 28 ft (8.5 m) wide. Reinforcing was exposed in all areas of deepest erosion. The back sides of upstream baffles 3, 4, and 5 were badly eroded.

The erosion of the concrete surfaces was attributed to high-velocity discharges reworking a large volume of gravel. The gravel was deposited over 30 years from the downstream riverbed by eddy currents. Approximately, 300 cu yd (229 m^3) of gravel was removed during a 1983 inspection.

Repair

In 1984, the stilling basin floor was overlaid by Corps of Engineers personnel with a conventional 5,000-psi (34.5-MPa) concrete having a 19.0-mm (3/4-in.) nominal maximum size aggregate. During the dewatering of the stilling basin, another 40 cu yd (31 m^3) of gravel (deposited over a 1-year period) was found in the stilling basin and removed. The downstream riverbed was redressed smooth with a bulldozer. The removal and bulldozer work was completed in about 8 hr.

The repair area perimeter was saw cut 2 in. (51 mm) deep (Figure 2). Jackhammers were used to remove concrete within the perimeter a minimum of a 2-in. (51-mm) depth. The damaged areas located between the upstream and downstream rows of baffles were too confined to allow saw cutting the perimeter. In these areas, the perimeter edge was created by line drilling overlapping holes with jackhammers. The 40-year-old concrete was hard, especially the aggregate. The area was thoroughly cleaned and hand inspected for loose concrete. All exposed reinforcing bars that were undercut had concrete removed from underneath to allow 2 in. (51 mm) of concrete placement. Final cleanup was by a contractor using a 1,500-psi (10.3-MPa) steam water blaster. A total of 19 hr was required to prepare surfaces.

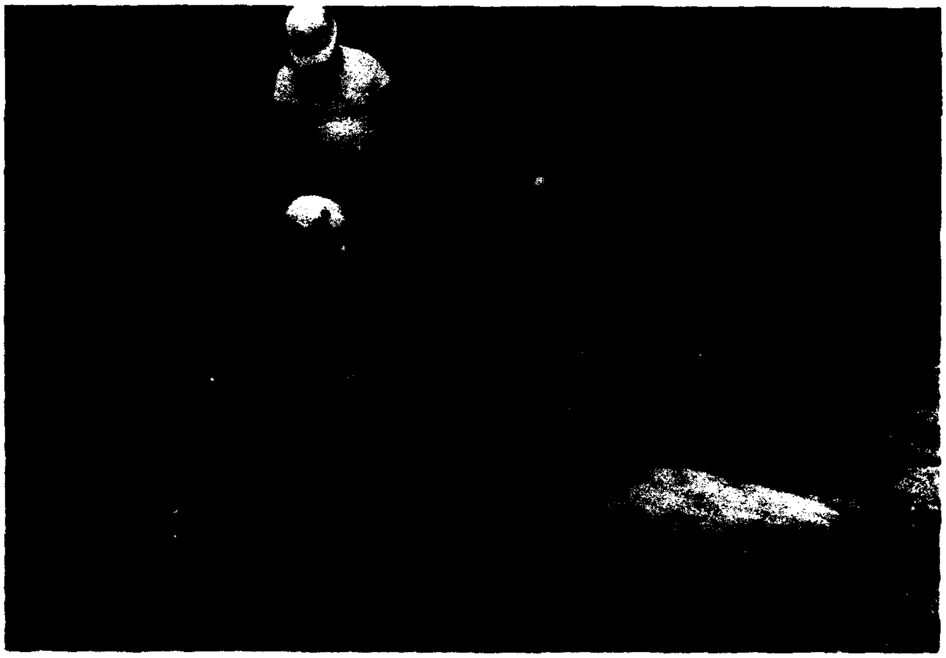

Figure 2. Saw cutting of repair perimeter, Clearwater Dam Stilling Basin, 1984

The prepared surfaces were coated with a two-component epoxy, bonding agent (Type 1-215, HM, Grade 2, Class B, Epoxy manufactured by Permagile-Salmon, Ltd.). The epoxy was mixed in 5-gal (18.9-L) units and applied to the surface in 1-gal (3.8-L) buckets with holes drilled in the bottoms (Figure 3). The epoxy was evenly spread across the rough concrete surface by broom.

After the epoxy bond became tacky, the overlay concrete was placed and finished (Figure 4). The surface was sprayed with curing compound after finishing of concrete (Figure 5). The placement, finishing, and initial curing of the concrete were performed over a period of about 4 hr. The construction joints were sawed the next day.

The total cost for the repair was about $54,000. An estimated 110 yd^2 (92 m^2) of floor was resurfaced at total unit cost of \$500.00/yd^2 (\$598/m^2), including dewatering and removal costs.

Performance

Although redeposits of gravel into the stilling basin from downstream channel are expected to continue, the results of underwater video inspections in 1986, 1987, and 1990 showed the overlay to be in good condition. The next inspection is scheduled for September 1993.

DeQueen Dam Stilling Basin

Background

The DeQueen Dam was completed in 1977 and is located in the Little Rock District on the Rolling Fork River near DeQueen, AR. The dam is a rolled earthfill embankment with a concrete outlet works and an uncontrolled emergency spillway. The outlet works of the dam were completed in 1970, and the first diversion through the stilling basin took place in 1973 before the concrete fishing berms were constructed on both sides of the downstream channel. The drop section and stilling basin of the outlet are U-framed in design and have a total length of 157.5 ft (48.0 m) and a width of 35 ft (10.7 m). The stilling basin is 77.5 ft (23.6 m) long and has a floor elevation which is 14 ft (4.26 m) below the invert of the conduit. The stilling basin contains two rows of baffle blocks (total of nine) and an end sill both of which extend 4 ft (1.22 m) above the stilling basin floor.

During a inspection in 1981, abrasion erosion damage was observed in the stilling basin floor both upstream and downstream of baffle blocks. The floor upstream of the baffles had an area 18 ft (5.49 m) long by 18 bars wide of exposed reinforcing bars. Rounded rocks were observed in holes where reinforcing were exposed.

Figure 3. Application of epoxy bond coat, Clearwater Dam Stilling Basin, 1984

Figure 4. Concrete finishing, Clearwater Dam Stilling Basin, 1984

Figure 5. Application of curing compound, Clearwater Dam Stilling Basin, 1984

It was concluded that the public was pushing, rolling, and throwing riprap into the basin from the locations on the high concrete ledges on the left and right downstream banks. Fishermen positioned riprap stones along the ledges to use as seats and then rolled stones into the stilling basin when no longer needed.

Repair

In 1983, the stilling basin floor was overlaid with approximately 2,900 yd^2 (2,435 m^2) of 12-in. (305-mm)-thick, reinforced concrete. The reinforcement (Figure 6) consisted of no. 6 (19-mm) bars on 12-in. (305-mm) centers in both the longitudinal and transverse directions and no. 4 (13-mm) bar vertical dowels on 2-ft (0.61-m) centers in both directions. A nonshrink grout was specified to secure dowels. An epoxy grout was specified as a bonding agent between the stilling basin concrete and the overlay. The overlay concrete had a specified compressive strength of 3,000 psi (20.7 MPa).

Other changes to the stilling basin included modifying the baffle blocks to maintain difference in elevation between floor and top of blocks; changing orientation of the upstream face of the end sill from vertical to 45 deg; and extending the concrete channel 80 ft (24.4 m) downstream of end sill.

Figure 6. Prepared surface with reinforcement in place, DeQueen Dam Stilling Basin, 1983

Performance

An underwater video inspection in 1987 revealed that abrasion erosion had occurred along two-thirds of the construction joint between the ogee and stilling basins sections and around the upstream row of baffles. The most extensive damage along the joint was 6 in. (152 mm) deep and around the baffle blocks was 10-in. (254 mm) deep. Exposed reinforcing bars were observed in a number of eroded areas around baffle blocks. Shrinkage cracks were observed at random in large pattern arrangement (Figure 7).

The eroded areas were patched during unwatered conditions by Corps of Engineers personnel in 1989. Concrete was removed using jackhammers to provide a 2-in. (51-mm) minimum patch depth, surfaces were brushed with an epoxy binder (Sikadur 32, Hi-Mod Epoxy Bonder), and 5,000-psi (34.5-MPa) concrete containing 3/4-in. (19-mm) nominal maximum size aggregate was placed. Riprap in areas adjacent to the stilling basin and downstream sidewalk were grouted in place to secure riprap against being moved by the general public.

Underwater video inspections in 1990 and 1991 indicated remedial patching currently still intact. The next inspection is scheduled for June 1992.

Figure 7. Cracking in floor, DeQueen Dam Stilling Basin, 1989

Emsworth Locks

Background

Emsworth Locks are located in the U.S. Army Engineer District, Pittsburgh, on the Ohio River 6.2 miles (10.0 km) downstream of the point of origin of the river at Pittsburgh, PA. The project was completed in 1922 and includes two parallel navigation locks; one is 600 ft (183 m) long by 110 ft (33.5 m) wide, and the other is 360 ft (110 m) long by 56 ft (17.1 m) wide. The concrete used to construct the locks was not air-entrained. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated.

During the first periodic inspection in 1972, the land, middle, and river wall monoliths were in an advanced stage of deterioration which included spalled areas, numerous random cracks, and signs of extreme weathering.

A polymer mortar was used to resurface the top surface of two of the monoliths (McDonald 1987). The resulting overlays were in poor condition with extensive pattern cracking after 1 year in service.

Repair

As a part of the 1982-1986 rehabilitation of the locks and dam, the tops of all three lock walls were overlaid with 12 in. (305 mm) of reinforced concrete. The concrete specifications required a 37.5-mm (1-1/2-in.) nominal maximum size aggregate, air content of 3.5 to 6.5 percent, a 2- to 5-in. (51- to 127-mm) slump, and a 28-day compressive strength of 3,000 psi (20.7 MPa). The reinforcement consisted of a mat of no. 5 (16-mm) bars on 12-in. (305-mm) centers in both the longitudinal and transverse directions. No. 6 (19-mm) bar dowels were installed on 24-in. (610-mm) centers in both the longitudinal and transverse directions.

The cost for overlaying the tops of lock walls excluding removal and reinforcement cost was $\$135/yd^2$ ($\$161/m^2$). The estimated overlay area was 2,900 yd^2 (2,425 m^2).

Performance

Cracking in the overlay was noted during the third periodic inspection in 1986. Cracks were observed along gate anchor locations in both the land and river wall overlays. Transverse cracks were noted in many of the monoliths indicating the need for more construction joints. Heavy pattern cracking was observed on several monoliths.

Montgomery Locks

Background

Montgomery Locks are located in the Pittsburgh District on the Ohio River 31.7 miles (51.0 km) downstream of the point of origin of the river at Pittsburgh, PA. The project was completed in 1936 and includes two parallel navigation locks; one is 600 ft (183 m) long by 110 ft (33.5 m) wide and the other 360 ft (109.7 m) long by 56 ft (17.1 m) wide. The concrete used to construct the locks was not air-entrained. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated.

Repair

The tops of all three lock walls were resurfaced during the 1985-1989 rehabilitation of the locks and dam with 12 in. (305 mm) of reinforced concrete. The concrete specifications required a 37.5-mm (1-1/2-in.) nominal maximum size aggregate, air content of 3.5 to 6.5 percent, a 2 in. to 5-in. (51- to 127-mm) slump, and a 28-day compressive strength of 3,000 psi (20.7 MPa). The reinforcement consisted of a mat of no. 5 (16-mm) bars on 12-in. (305-mm) centers in both the longitudinal and transverse directions. A no. 5 (16-mm), 4-ft (1.2-m) long reinforcing bar was placed diagonally at all

reentrant corners of blockouts to limit cracking of the overlay in this area. No. 6 (19-mm) bar dowels were installed on 24-in. (610-mm) centers in both the longitudinal and transverse directions.

The cost of overlaying the tops of lock walls excluding removal and reinforcement cost was $\$130/yd^2$ ($\$155/m^2$). The estimated area resurfaced was of 4,325 yd^2 (3,616 m^2).

Performance

Cracking in the overlay was noted during the fourth periodic inspection in 1989. For the most part, the overlay was in good condition with only limited development of cracks, primarily transverse. Exceptions included the gate monoliths where cracking was more extensive and at corners of blockouts where stress concentrations occur. A considerable amount of cracking occurred in the land wall overlay with cracks extending from almost all corners of blockouts, check posts, rail posts, and corner protection joints. Additionally, several sections developed pattern cracking over an entire slab.

Tuttle Creek Dam Stilling Basin

Background

The Tuttle Creek Dam was completed in 1962 and is located in the U.S. Army Engineer District, Kansas City, on the Big Blue River near Manhattan, KS. The outlet works of the dam consists of a tainter-gate controlled spillway and spillway bridge, twin horseshoe conduits, and twin stilling basins with a diversion wall. The stilling basin was constructed using 3,000-psi (20.7-MPa) concrete containing 467 lb/cu yd (277 kg/cu m) of cement and a 3-in. (76-mm) nominal maximum size aggregate. The coarse aggregate was made up of mostly Farley limestone and the fine aggregate, Big Blue River natural sand.

During a periodic inspection in 1967, abrasion erosion damage was observed in the transition slab and the upstream portion of the stilling basin slab. In four areas, the reinforcing steel was exposed with the largest area being approximately 10 ft (3.05 m) by 12 ft (3.66 m). The erosion was caused by rocks and other debris moved by eddy currents at low discharge flows. Temporary repairs were made in the areas with exposed aggregate.

Additional erosion damage was observed during the second periodic inspection in 1973. The reinforcing was exposed at the perimeter of patches but not within areas patched in 1967.

Repair

In 1975, the stilling basin and transition slabs were overlaid with a 10-in. (254-mm)-thick concrete containing an abrasion-resistant, Sioux Quartzite as the coarse aggregate. The mixture design required a Type II portland cement, 3/4-in. (19-mm) nominal maximum size aggregate, and a 28-day compressive strength of 6,000 psi (41.4 MPa). Coarse aggregate was required have a Los Angeles abrasion loss not to exceed 40 percent (American Society of Testing Materials (ASTM) C 131 (ASTM C 1988d)).

The concrete was batched using the following mixture proportions per cubic yard (cubic metre):

Portland cement	658 lb	(391 kg)
Coarse aggregate	1,772 lb	(1,052 kg)
Sand	1,166 lb	(691 kg)
Water	243 lb	(144 kg)

Test averages for batched concrete included an air content of 5.4 percent, a slump of 2-3/4 in. (70 mm), and a 28-day compressive strength of 5,130 psi (35.4 MPa).

Anchor bolts were provided between the existing slab and the overlay. The design considered that if lack of bond occurred over a several-square-foot (0.18 m^2 or more) area, the resistance to the uplift forces would be concentrated at the perimeter of this area and could be large enough to separate the overlay slab from the existing slab. The anchors were designed to carry the potential uplift forces.

Expansion anchor bolts having a 5/8-in. (16-mm) diameter were installed to a minimum depth of 7 in. (178 mm) in holes drilled in the existing concrete surface (Figure 8). A welded-wire fabric was placed above anchor bolts. The fabric had a steel area of 0.12 $in.^2/ft$ (254 mm^2/m) each way and a 4-in. (102-mm) opening.

During the placement of the overlay, the caulking of monolith joints in the stilling basin slab was not adequate for preventing seepage. As a result, water ponded on bonding grout that was being applied ahead of concrete placement. The 4-in. (102-mm) mesh opening made application of grout and removal of ponded water more difficult. Therefore, it is suspected that the bond strength between the overlay and the existing slab is less than intended.

The approximate area overlaid was 2,100 yd^2 (1,756 m^2). The costs for the repair are shown in Table 1.

Performance

The overlay was inspected in 1985, and it was in very good condition. Most of the overlay surface has exposed aggregate. Some areas of the

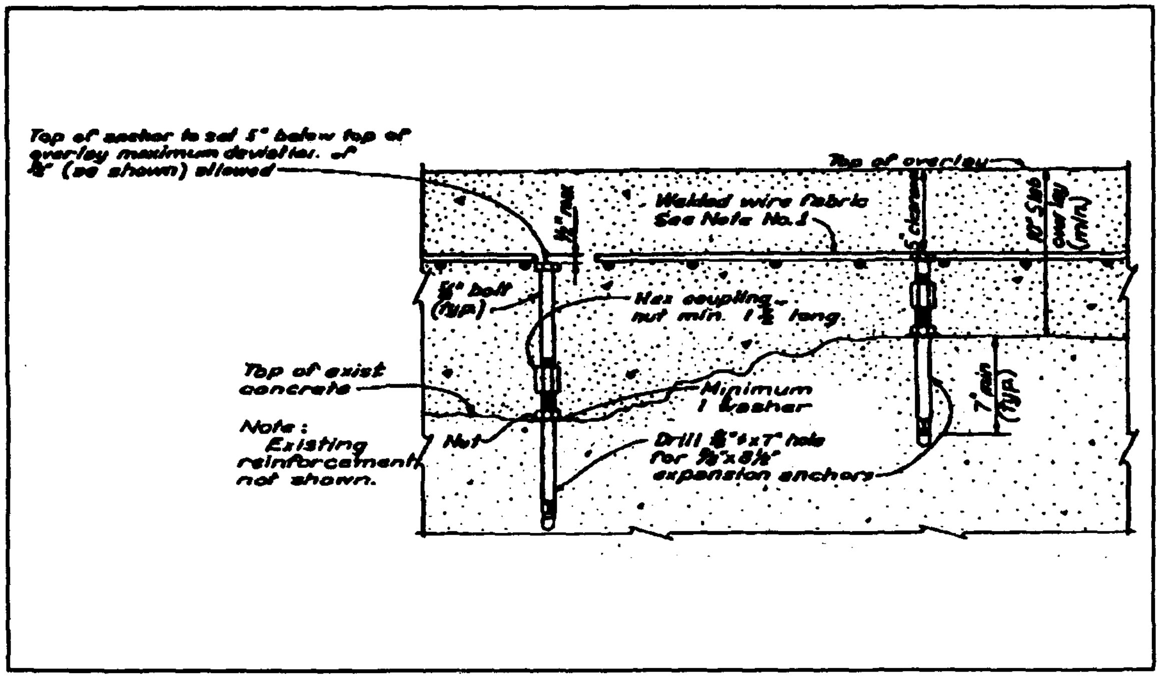

Figure 8. Overlay and anchor bolt details, Tuttle Creek Dam Stilling Basin, 1975

Table 1
Repair Cost, Tuttle Creek Stilling Basin, 1975

Item	Unit Cost $/yd², $/m²	Cost $
Dewatering	18 (22)	37,900
Concrete removal	2 (2)	5,000
Overlay	51 (61)	112,000
Reinforcing steel	2 (2)	5,500
Anchor bolts	17 (20)	36,200
Drain extensions, saw cutting	1 (1)	1,400
TOTALS	91 (109)	198,000

transition showed about a 2-in. (51-mm) difference in elevation from previous surveys. Possibly, this was because of the difficulty in surveying the same points on the transition slab between surveys. A few cracks were observed in the basin floor slabs. An inspection in 1990 also found the overlay to be in good condition.

3 Low-Slump Concrete Overlays

Mississippi River Lock 17

Background

Mississippi River Lock 17 is located in the U.S. Army Engineer District, Rock Island, on the Mississippi River near New Boston, IL. The project was completed in 1936 and includes a 600-ft (183-m) long by 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and contained a crushed limestone coarse aggregate. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated. Some nondurable aggregates were also present that increased the extent of deterioration.

Repair

In 1989 and 1990, the top surfaces of the lock walls were overlaid using *low-slump concrete mixture to which a water-reducing admixture* had been added. The concrete specifications required a Type I portland cement, 19.0-mm (3/4-in.) nominal maximum size aggregate, air content of 6.0 to 7.5 percent, maximum slump of 3 in. (76 mm), and a 28-day compressive strength of 4,000 psi (27.6 MPa). The concrete was batched using the following approximate mixture proportions per cubic yard (cubic metre) of concrete:

Cement	823 lb (488 kg)
Coarse aggregate (crushed limestone)	1,396 lb (828 kg)
Fine aggregate (natural sand)	1,396 lb (828 kg)
Water (36 gal, 137 L)	300 lb (178 kg)
Water-reducing admixture	As required
Air-entraining admixture	As required

Gomaco Scaraplane 1 (cold planer, similar to the one shown in Figure 28) was used to remove 4 in. (102 mm) of concrete from top surface of lock walls. Hand-held breakers were used to remove the concrete in congested

areas and in areas where steel reinforcement was expected. The surface was cleaned by sandblasting and waterblasting. The surface was kept moist for a minimum of 12 hr prior to concrete placement. The surface was given a broom finish and cured for 7 days using wet burlap.

The concrete removal cost was $105/yd^2 ($126/m^2), and the concrete replacement cost was $44/yd^2 ($53/m^2). The estimated resurfaced area was 3,510 yd^2 (2,930 m^2).

Performance

Minor cracking was observed in the overlay during an August 1991 visit to the project. Cracking was most frequent at corners of blockouts. These cracks were random in direction and typically intersected other cracks that were random in direction. In general, the cracks appeared to be wider and longer than similarly located cracks observed at Mississippi River Lock Number 21 where 4-in. (102-mm) thick polypropylene-fiber-reinforced concrete had been placed and at Mississippi River Lock Number 22 where latex-modified concrete had been placed.

Mississippi River Lock 18

Background

Mississippi River Lock 18 is located in the U.S. Army Engineer District, Rock Island, on the Mississippi River near Burlington, IA. The project was completed in 1937 and includes a 600-ft (183-m) long and by 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and contained a crushed limestone coarse aggregate. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated. Some nondurable aggregates were also present that increased the extent of deterioration.

Repair

The tops of lock walls were resurfaced in 1991 using a low-slump concrete mixture to which a water-reducing admixture had been added. The concrete specifications required a Type I portland cement, 19.0-mm (3/4-in.) nominal maximum size aggregate, air content of 6.0 to 7.5 percent, maximum slump of 3 in. (76 mm), and a 28-day compressive strength of 4,000 psi (27.6 MPa). The saturated surface dry batch weights per cubic yard (cubic metre) were specified as follows:

Cement	823 lb (488 kg)
Fine aggregate	1,398 lb (830 kg)
Coarse aggregate	1,398 lb (830 kg)
Water	287 lb (170 kg)
Water-reducing admixture	As Required
Air-entraining admixture	As Required

A cold planer similar to the one shown in Figure 28 was used to remove 4 in. (102 mm) of concrete from the top surface of lock walls. Hand-held breakers were used to remove concrete in congested areas and in areas where steel reinforcement was expected. Prior to removal, saw cuts were made along repair perimeter to reduce overbreakage.

The bond surfaces were cleaned with a 3,000-psi (20.7-MPa) water jet (Figure 9) and then sandblasted. Vertical faces were formed along outer boundaries of overlay and at machinery and other areas to be blocked out. Joints were formed in the overlay to coincide with lock monolith joints. Two-piece, hollow, plastic strips in which the narrower top half seats within the lower half were used to form rectangular blockouts for placement of joint sealant. The surface was kept moist for a minimum of 12 hr before concrete placement. A grout consisting of equal parts by mass of portland cement and fine aggregate was broomed into surfaces just ahead of concrete placement (Figure 10).

Concrete was delivered to job in transit mixers. For middle and river walls, concrete was placed in buckets and ferried to lock walls. Buckets were lifted to top of lock wall by barge crane and concrete discharged (Figure 11). Concrete was vibrated and surface screeded, hand-floated, and bull-floated to bring overlay to required finish level (Figure 12). A broom finish was applied to the surface and surface was wet cured for 7 days using wet burlap. After removal of burlap, the top halves of plastic strips were lifted out of joints and joint sealant placed in hollow portion of remaining strip.

Performance

Minor cracking was observed in the overlay during the summer of 1991. Cracking was most frequent at corners of blockouts. These cracks were random in direction and typically intersected other cracks that were random in direction. Random hairline cracking was indicated in one section by the discoloration of the surface (Figure 13). This section was placed on a hot summer day and curing compound applied to the surface. It is hypothesized that cracking developed in the overlay just prior and during the time in which the curing compound was applied, and that as the cracks propagated, the curing compound flowed into the crack openings leaving the surface bordering the cracks with little compound and discolored.

Figure 9. Bond surface cleaned with a water jet, Mississippi River Lock 18, 1991

Figure 10. Bond grout brushed into surface, Mississippi River Lock 18, 1991

Figure 11. Placing of concrete on the top of the lock wall, Mississippi River Lock 18, 1991

Figure 12. Screeding and floating of placed concrete, Mississippi River Lock 18, 1991

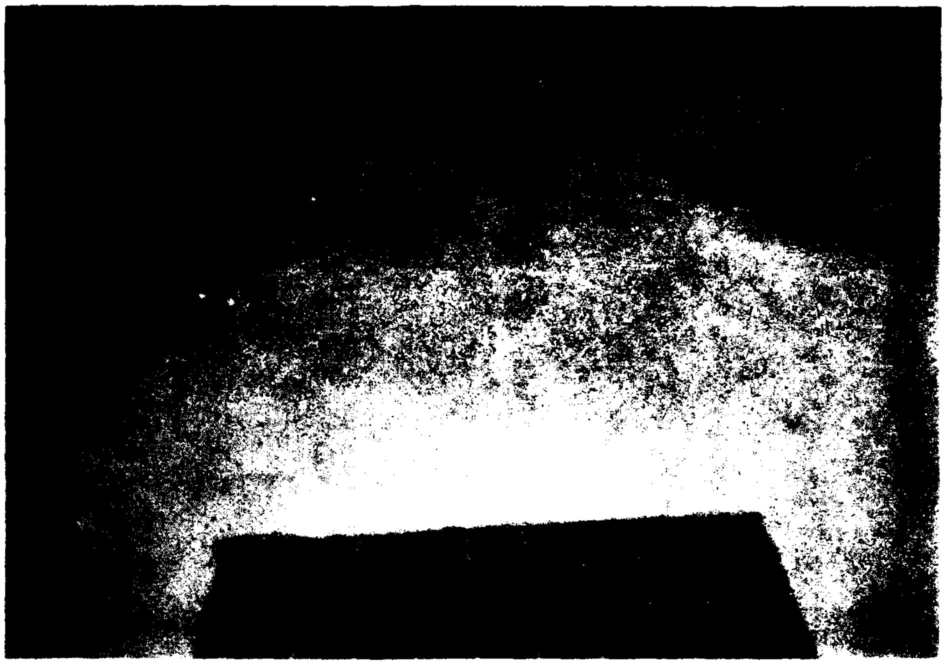

Figure 13. Discolorations in surface indicate locations of hairline cracks, Mississippi River Lock 18, 1991

Mississippi River Lock 20

Background

Mississippi River Lock and Dam 20 is located in the Rock Island District on the Mississippi River near Canton, MO. The project was completed in 1935 and includes a 600-ft (183-m) long and 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and contained a natural gravel coarse aggregate composed predominantly of igneous rock types. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated. Some nondurable aggregate particles were also present that increased the extent of deterioration.

Repair

In 1987, the top surfaces of the lock walls were resurfaced using a low-slump concrete mixture to which a water-reducing admixture had been added. The concrete specifications required a Type I portland cement, 19.0-mm (3/4-in.) nominal maximum size aggregate, air content of 6.0 to 7.5 percent, maximum slump of 3 in. (76 mm), and a 28-day compressive strength of 4,000 psi (27.6 MPa). The concrete was batched using the following approximate mixture proportions per cubic yard (m^3) of concrete:

Cement	823 lb (488 kg)
Fine aggregate (crushed aggregate)	1,398 lb (830 kg)
Coarse aggregate (natural sand)	1,398 lb (830 kg)
Water	287 lb (170 kg)
Water-reducing Admixture	As Required
Air-entraining admixture	As Required

A cold planer similar to the one shown in Figure 28 was used to remove 4 in. (102 mm) of concrete from top surface or lock walls. Hand-held breakers were used to remove concrete in congested areas and in areas where reinforcement was expected. The concrete surface was cleaned by wet sandblasting and water jet blasting. The surface of the concrete was keep wet for 12 hr immediately before placement. A bond grout was broomed into the surface prior to placement of the concrete. The concrete was consolidated using a small vibrator and a vibrating screed. The concrete was finished using a bull float and a stiff broom. The concrete was moist cured using wet burlap covered by polyethylene sheet.

Considerable difficulty was encountered in removing the original concrete. As a result, the contractor's cost for removing the concrete was much greater than his bid price. The contractor filed a claim against the government based on "differing site conditions" and was awarded compensation. The "differing site conditions" were that the concrete was sounder and the aggregate much harder than the concretes removed by the contractor from similar Corps of Engineers projects in the district.

Prior to compensation award, the concrete removal cost was $85/yd^2 ($102/m^2) and the concrete replacement cost, $81/yd^2 ($97/m^2). The estimate area for overlay was 2,340 yd^2 (1,960 m^2).

Performance

Minor cracking was observed in the overlay during a periodic inspection in August 1991. Cracking was most frequent at corners of blockouts (Figure 14). These cracks were random in direction and typically intersected other cracks that were random in direction (Figure 15). Many of the cracks in the overlay concrete were shrinkage cracks that reflected cracks in the underlining concrete.

Red Rock Dam Spillway Bridge

Background

Red Rock Dam was completed in 1969 and is located in the Rock Island District on the Des Moines River about 142 miles (228 km) upstream from point of discharge into the Mississippi River. The nearest cities are Pella and Knoxville, about 4 and 6 miles northeast and southwest, respectively. The

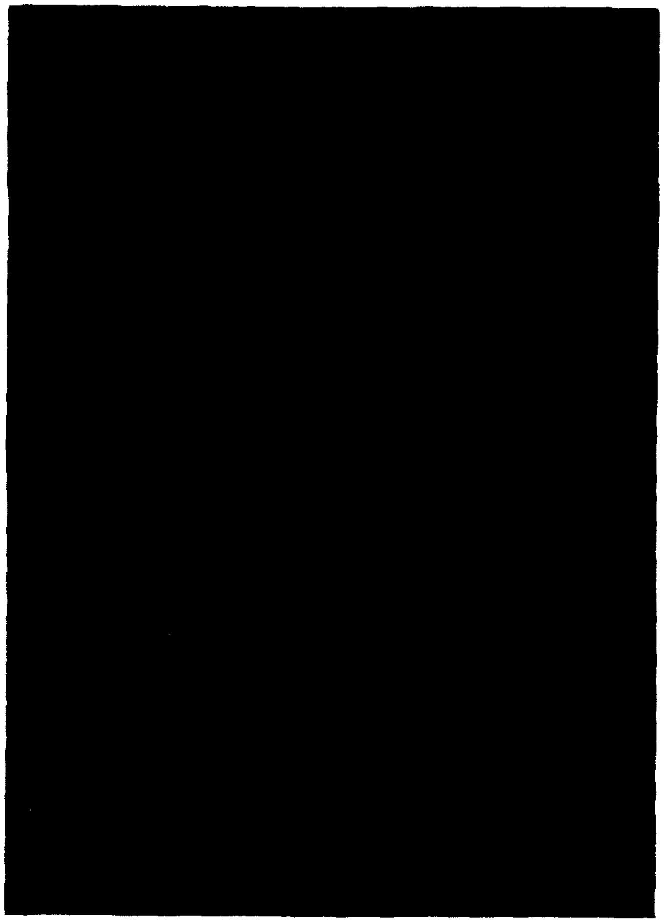

Figure 14. Cracking typically found at corners of blockouts, Mississippi River Lock 20, 1991

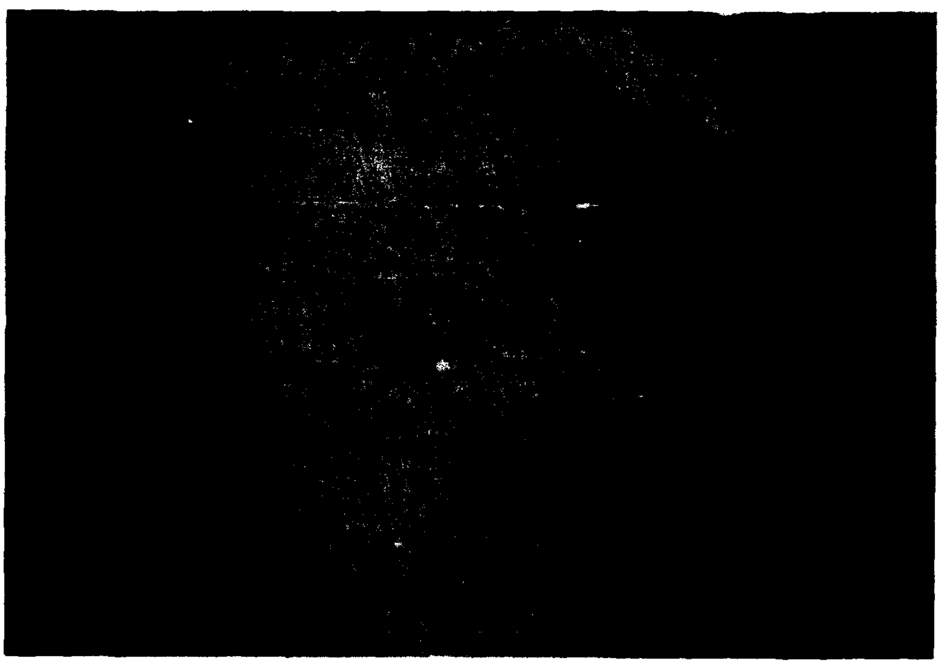

Figure 15. Random cracking typically found in wide monoliths, Mississippi River Lock 30, 1991

overflow section of the dam includes a 28-ft (8.5-m) wide by 563-ft (172-m) long bridge having a deck thickness of about 6 in. (152 mm). The bridge is part of the route of Marion County Road T15 which connects Pella and Knoxville.

In 1980, spalling had occurred in the roadway surface of the spillway bridge to the extent that repair was needed (Figures 16 and 17). The spalling was the result of application of deicing salts and weathering. Epoxy mortar patches were used to maintain the roadway surface of the bridge until a permanent repair could be made. In 1981, a survey was conducted that located spalled and hollow (drummy) areas to be repaired.

Repair

In 1982, a contract was let to resurface the bridge roadway surface using low-slump concrete to which a water-reducing admixture had been added. The overlay thickness was 1-3/4 in. (44 mm) which increased the deck thickness by 1-1/2 in. (38 mm). The work involved patching approximately 600 yd^2 (502 m^2) of roadway surface followed by overlaying the entire bridge roadway.

During the repair, the one-way traffic was maintained across the bridge. The roadway surface was removed to a depth of 1/4 in. (6 mm) below the existing finished surface. In areas to be patched, the concrete was removed to the level of the top reinforcing steel. In areas where the deterioration was more extensive, the concrete was removed below the level of the top reinforcing to sound concrete. The bond surfaces were sand-blasted and were cleaned with air immediately prior to applying the bond grout. The bond grout consisted of equal parts by weight of portland cement and fine aggregate. The grout was broomed into surfaces just prior to placement of concrete.

The concrete batch specifications required an air content of 6.5 percent plus or minus 1.0 percent and a maximum slump of 1.0 in. (25 mm). The saturated surface dry batch weights per cubic yard (cubic metre) were approximately as follows:

Cement	823 lb (488 kg)
Fine aggregate	1,394 lb (827 kg)
Coarse aggregate	1,394 lb (827 kg)
Water-reducing admixture	As required
Air-entraining admixture	As required

These quantities were based on the specific gravity of the cement being 3.14 and the fine and coarse aggregates being 2.65.

The finished concrete was cured using wet burlap for 72 hr. The burlap was applied as soon as the surface would support it without deformation.

Figure 16. Spalling of bridge roadway, Red Rock Dam Spillway Bridge, 1980

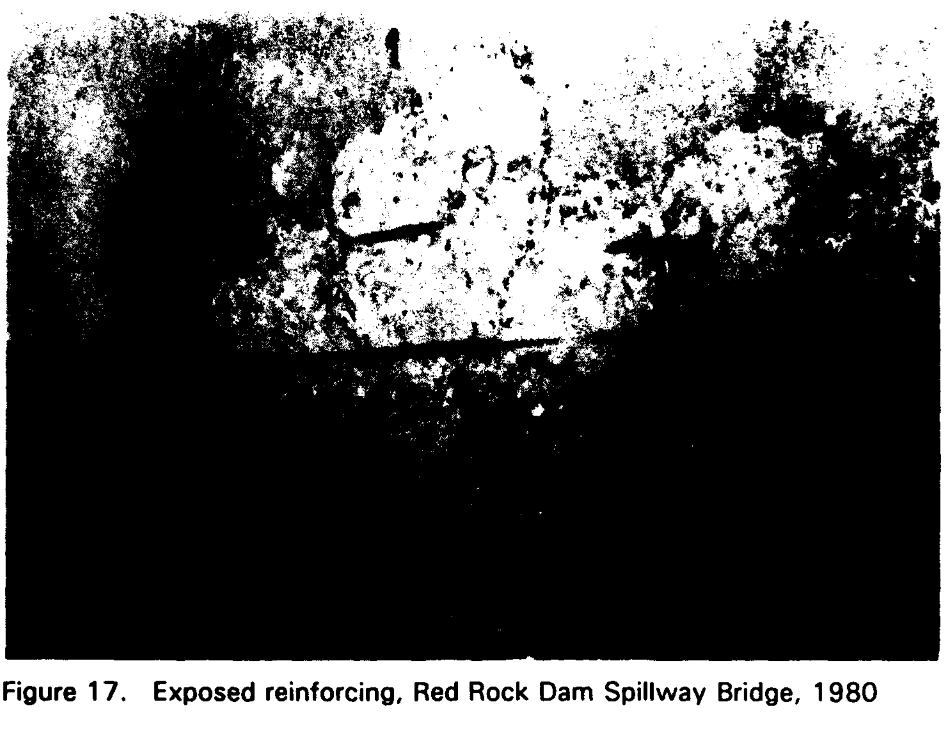

Figure 17. Exposed reinforcing, Red Rock Dam Spillway Bridge, 1980

After curing, linseed oil was applied to the concrete surfaces to delay spalling. Figure 18 shows completed roadway surface.

Figure 18. Completed overlay, Red Rock Dam Spillway Bridge, 1982

The contract period was for 300 days at a cost of \$68,244. The cost for concrete removal was \$28/yd^2 (\$33/m^2) and concrete replacement cost, \$33/yd^2 (\$39/m^2). The estimated resurfaced area was 1,815 yd^2 (1,518 m^2).

Performance

The bridge deck overlay was inspected in May, 1987 and was in excellent condition with minor shrinkage cracks noted.

Tuttle Creek Dam Spillway Bridge

Background

The Tuttle Creek Dam was completed in 1962 and is located in the Kansas City District on the Big Blue River near Manhattan, KS. The outlet works of the dam consist of a tainter-gate controlled spillway, twin horseshoe conduits, and twin stilling basins with a diversion wall. A 18-span bridge was constructed over the spillway. The roadway surfaces of the bridge deck spans had deteriorated as a result of applications of deicing salts and subsequent chloride-ion penetrations that caused corrosion of reinforcing steel.

Repair

The bridge deck was repaired in 1987 by replacing the most severely deteriorated slabs and overlaying the remaining slabs with a 2-1/4-in.(57-mm)-thick, low-slump concrete. The concrete overlay required a Type I or II, low-alkali, portland cement, 19.0-mm (3/4-in.) nominal maximum size aggregate, air content of 4.0 to 7.0 percent, maximum slump of 1 in. (25 mm), and a 28-day compressive strength of 4,000 psi (27.6 MPa). The cement content was 823 lb/cu yd (488 kg/cu m). The fine aggregate was 50 percent of the total aggregate by absolute volume. Coarse aggregate was required have a Los Angeles abrasion loss not to exceed 40 percent (ASTM C 131 (ASTM 1988d)).

The roadway of bridge decks was scarified to remove a minimum of 1/4-in. (6-mm) of surface. Approximately 20 yd^2 (16.7 m^2) of roadway had to be patched of which 4 yd^2 (3.34 m^2) required full depth patches. Details for patching are presented in Figure 19. A 3/4-in. (19-mm) clearance was provided between exposed reinforcing bars and concrete. Reinforcing was cleaned by sandblasting. Cut or broken bars or bars in which 10 percent or more of cross-sectional area had been lost were spliced with a new bar of the

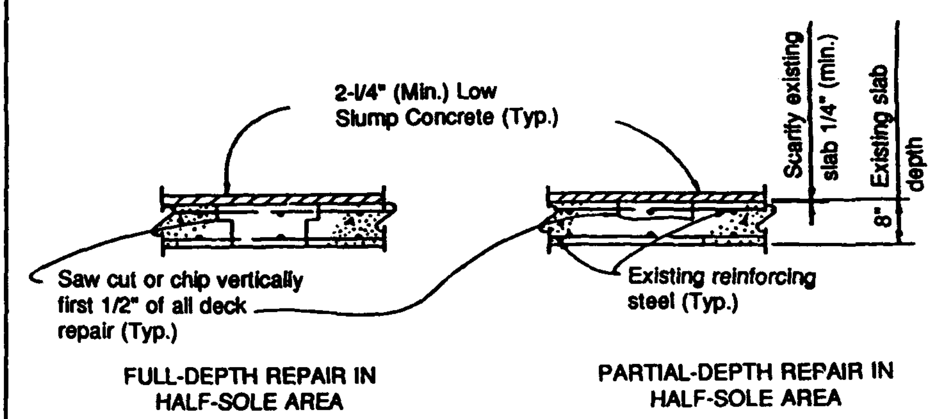

Figure 19. Roadway patching details, Tuttle Creek Dam Spillway Bridge, 1987

same size, 34 in. (864 mm) long for top bars and 24 in. (610 mm) for bottom bars.

A thin coat of bonding grout was scrubbed into the dry prepared surfaces just prior to concrete placement. A total of 1,900 yd^2 (1,590 m^2) of overlay was placed. As a part of finishing, overlay surfaces were given a rough texture while concrete was still plastic. Surfaces were wet cured for 72 hr. Overlay was kept out of service until concrete compressive strength reached 3,000 psi (20.7 MPa).

Performance

A 1989 inspection of the bridge deck found the overlay to be in good condition.

4 Fly-Ash Concrete Overlays

Los Angeles River Main Channel

Background

The Los Angeles River is located in the U.S. Army Engineer District, Los Angeles, and flows through the city of Los Angles, CA, draining a 753-sq-mi (1,950-km^2) watershed. In the 1940's, the majority of the 50-mile (80-km) long river channel was constructed and paved with 12-in. (305-mm) thick, conventional reinforced concrete. During the passing years, various features of the concrete channel had deteriorated or been damaged. The most significant damage occurred within a 12-mile (19.3-km) reach of the low-flow channel as the result of abrasion erosion caused by water-borne sediment. In some reaches, the erosion had progressed completely through the concrete slab. Minor abrasion erosion damage also occurred in the main channel invert.

Repair

The main channel was overlaid during 1986 through 1988 using unreinforced fly-ash concrete. The overlays were placed 6-in. (152-mm) thick. The existing concrete at some of the joints required patching. Damaged areas were saw cut a minimum of 1 in. (25.4 mm) outside the perimeter of damage and concrete within was removed by chipping. Depths of saw cuts were 3 in. (76 mm) for slabs and 2 in. (51 mm) for walls. Channel surface was shotblasted (Figure 20) prior to placement of concrete.

The overlay concrete was specified to have a Type II cement, Class F fly ash, nominal maximum size aggregate of 37.5-mm (1-1/2 in.), and slump between 2 and 5 in. (51 and 127 mm). No air-entraining admixture was specified. The mortar portion of the mixture was applied ahead of the placement of the overlay as a bond coat. The concrete mixture proportions were modified throughout the rehabilitation and are presented in Table 2. The average compressive strength of these mixtures was 5,000 psi (34.5 MPa) at 28-day age.

Figure 20. Shotblasting in preparation for overlay, Los Angeles River Main Channel, 1987

Table 2
Mixture Proportions for Los Angeles River Main Channel

	Weight, lb/cu yd (kg/cu m)		
Item	**1986**	**1987**	**1988**
Portland cement	570 (338)	529 (314)	586 (348)
Fly ash	140 (83)	120 (71)	100 (59)
Coarse aggregate	1,905 (1,131)	2,020 (1,199)	2,035 (1,208)
Fine aggregate	1,163 (690)	1,220 (724)	1,130 (671)
Water	287 (170)	258 (153)	270 (160)
Water-cement ratio by mass	0.40	0.40	0.39

The total cost for the fly-ash concrete overlays was \$21/yd^2 (\$25/m^2). The estimated resurfaced area was 733,000 yd^2 (613,000 m^2). For comparison purposes, the total cost for the silica-fume concrete overlay used to rehabilitate the Los Angeles River Low-Flow Channel (1983-1985) was \$28/yd^2 (\$33/m^2). The estimated resurfaced area was 160,000 yd^2 (134,000 m^2).

Performance

Inspections of the channel have occurred during the construction of the last phases and subsequent to the last rehabilitation contract and the initial performance of the overlays was judged to be excellent. The next scheduled inspection is for fiscal year 1994.

Mojave River Forks Dam Outlet Tunnel

Background

The Mojave River Forks Dam is an earthfill flood control structure that is located in the Los Angeles District on the West Fork of the Mojave River in San Bernardino County, California. The outlet tunnel through the dam is a concrete lined, ungated, horseshoe shaped structure that was completed in 1969.

Sediment transported by flows up to 40 ft/sec (12.2 m/sec) had eroded an average of 4 to 6 in. (102 to 152 mm) of concrete from the invert of the tunnel. In some areas, the reinforcing steel was exposed and in some cases missing. The original cover for the reinforcing was 6 in. (152 mm). The sediment abrading the concrete consisted of cobbles, gravel, and very coarse sands.

Repair

A variety of overlay concretes were investigated for the repair. These included polymer-modified, silica-fume, fly-ash, and polymer concretes. Based on the cost of construction, remoteness of the site, and anticipated project life, a 6,000-psi (41.4-MPa), fly-ash concrete was specified. The repair consisted of overlaying existing tunnel floor to an elevation of 1 ft (0.305 m) above original floor elevation (Figure 21). The repair was begun in August and completed in November of 1990.

The existing concrete surface was prepared by high-pressure water jet blasting using a 20,000-psi (138-MPa) working pressure and a flow rate of less than 100 gpm (378 L/min).

Existing Tunnel Dimensions, ft (m)				Elevation 'A' (Top of Overlay), ft (m)
Station	F	R	W	
2+00 (60.96 +00)	2.5 (0.76)	15.83 (4.82)	31.66 (9.65)	2,989.0 (911.05)
		VARIES	VARIES	VARIES
2+19 (60.96 +5.79)	2.5 (0.76)	9.5 (2.90)	19 (5.79)	2,988.5 (910.89)
11+74.23 (335.23 +22.63)	2.5 (0.76)	9.5 (2.90)	19 (5.79)	2,963.52 (903.28)

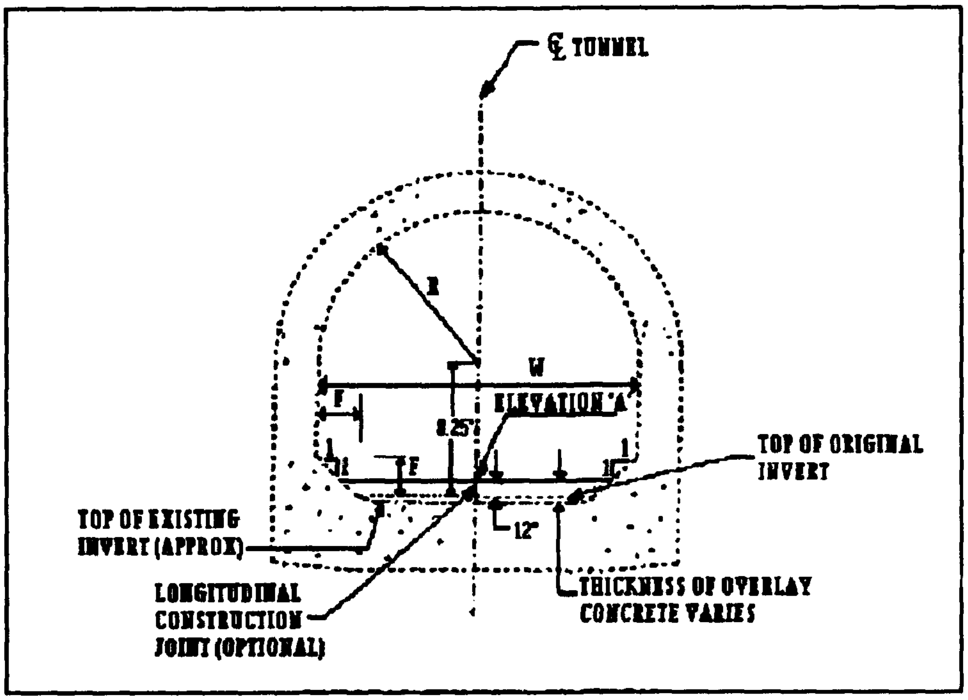

Figure 21. Typical tunnel section for overlay repair, Mojave River Forks Outlet Tunnel, 1990

The concrete batched contained a Type II portland cement and a Class F fly ash and had an average slump of 7 in. (178 mm), air content of 1.25 percent, and a compressive strength of 7,000 psi (48.3 MPa) at 28-day age. The concrete was batched using the following mixture proportions per cubic yard (cubic metre) of concrete:

Cement (Type II)	640 lb (380 kg)
Fly ash (Class F)	112 lb (66 kg)
Sand	1,306 lb (775 kg)
Coarse aggregate (ASTM C 33 (1988b), size number 3)	1,567 lb (930 kg)
Coarse aggregate (ASTM C 33 (1988b), size number 4)	250 lb (148 kg)
Water	267 lb (158 kg)

The concrete was transported approximately 40 miles (64.4 km) from the batch plant to the job site. To meet the maximum placement temperatures of 85 °F (29 °C), the contractor chilled water, introduced half the batch water as ice, and used a retarding admixture. With the addition of high-range water reducer at the batch plant and a discharge temperature of 65 °F at the job site, substantial delays in set occurred on the first day's placement. The tunnel acted as a wind tunnel to concentrate air flows, and as a result of the high wind velocities, plastic shrinkage cracking developed. The solution to prevent the cracking was to first decrease the batch ice by 50 percent and reduce the retarding admixture dosage. Subsequent results indicated that the concrete was being deposited at temperatures of 70 to 80 °F. Second, while concrete was being placed and until such time as the curing compound had been applied, at least one of the tunnel ends was covered with sheets to eliminate the wind tunnel effect. The results of these actions helped to eliminate the cracking.

The cost for surface preparation (water jet blasting) was $\$201/yd^2$ ($\$240/m^2$). The cost for overlay was $\$101/yd^2$ ($\$121/m^2$). The estimated overlay area was 1,650 yd^2 (1,380 m^2).

Performance

Observations by Operations Branch personnel indicate that the overlay is performing excellently after 1 year of service. However, no flows of the magnitude reported to have resulted in erosion damage to the tunnel floor have occurred since the repair. The next major periodic inspection was scheduled for the fall of 1993.

5 Silica-Fume Concrete Overlays

Kinzua Dam Stilling Basin

Background

Kinzua Dam is located in the U.S. Army Engineer District, Pittsburgh, on the Allegheny River approximately 198 miles (319 km) above the mouth of the river at Pittsburgh, PA. The project was completed in 1965 and includes a concrete gravity type spillway and stilling basin. The spillway contains four tainter-gate sections and an outlet works consisting of six low-level sluices through the spillway. The stilling basin is 204 ft (62.2 m) wide and 178 ft (54.3 m) long and contains nine 8-ft (2.4-m) high by 10-ft (3.0-m) wide by 18-ft 8-in. (5.7-m) long baffle blocks. The stilling basin floor was constructed a minimum of 5 ft (1.5 m) thick using concrete having a 6-in. (152-mm) nominal maximum size aggregate and a compressive strength of 3,000 psi (20.7 MPa) at 28-day age.

In 1969, abrasion erosion damage was observed in the stilling basin by scuba divers (McDonald 1980). Most of the damage was located at the contraction joints and corners of baffle blocks. Scour holes up to 42-in. (1,070-mm) deep and exposed reinforcing were also observed. By 1973, the erosion was present over most of the stilling basin floor.

In 1973-74, a steel-fiber-reinforced concrete overlay was used to repair and raise the stilling basin floor 1 ft in elevation (McDonald 1980). An underwater inspection by divers in 1977 indicated that erosion in the fiber-reinforced concrete overlay had reached a maximum depth of 36 in. (914 mm). After about 10 years of erosion damage, the stilling basin was again in need of repair. It was concluded that the fiber-reinforced concrete performed no better that the original concrete in resisting abrasion erosion (Holland and Gutschow 1987).

Repair

A silica-fume concrete overlay was used to repair the stilling basin floor in 1983 (Holland and Gutschow 1987). The Pittsburgh District had selected silica-fume concrete as the overlay material based on results from a U.S. Army Engineer Waterways Experiment Station (WES) study to find the highest abrasion resistance concrete mixture for the repair of Kinzua Dam stilling basin (Holland 1983).

The remaining fiber-reinforced concrete was removed from the stilling basin floor. Dowel holes were drilled and dowel bars installed to anchor the silica-fume concrete overlay to the existing concrete. The surface was prepared by wet sandblasting and high-pressure water jetting. A locally available limestone was used as the coarse aggregate. The concrete was specified to have a compressive strength of 12,500 psi (86.2 MPa), a maximum portland cement content of 700 lb/cu yd (415 kg/cu m), a minimum silica fume content of 15 percent by mass, and a maximum water-cement plus silica fume ratio by mass of 0.30 of the portland cement.

The concrete mixture proportions per cubic yard (cubic metre) were as follows:

Portland cement (Type I/II)	650 lb (386 kg)
Silica-fume-slurry admixture	
Silica fume	118 lb (70 kg)
Water	134 lb (80 kg)
Chemical admixtures	11 lb (6 kg)
Coarse aggregate	1,637 lb (971 kg)
Fine aggregate	1,388 lb (824 kg)
Water	85 lb (50 kg)

The water, aggregates, and cement were batched into transit mixer followed by the silica-fume-slurry admixture. The concrete was mixed for 5 min before being sent 8 miles (12.9 km) to the placement site. The concrete was pumped into forms (54 individual slabs). Two vibrating screeds (5 ft (1.5 m) apart) in tandem were used to vibrate and screed concrete. A curing compound was applied immediately after the second screed passed over the concrete.

As a part of the repair, a debris trap was constructed downstream of end sill of stilling basin to prevent debris from entering the basin from downstream. The trap was 25-ft (7.6-m) long with a 10-ft (3.0-m) high end sill that spanned the entire width of the basin.

The cost for overlaying the stilling basin floor excluding costs for mobilization, dewatering, removal, and filling holes was $\$90/yd^2$ ($\$108/m^2$). The total cost of repair was $\$394/yd^2$ ($\$471/m^2$). An estimated 3,440 yd^2 (2,880 m^2) of floor was overlaid.

Performance

Cracking in silica-fume concrete overlay appeared 2 or 3 days after placement (Holland and Gutschow 1987). The cracks were full depth and divided a slab into 5 or 10 portions. The cracks usually went through aggregate particles and were attributed to restraint of volume changes resulting from thermal expansion and contraction and autogenous shrinkage. No attempt was made to repair the cracks.

During an inspection in August of 1984, it was observed that about 1/2 in. (13 mm) in depth had been lost along some cracks and joints. In September of 1984, approximately 500 cu yd (380 cu m) of debris was removed from the basin, the debris trap, and the area immediately downstream of the trap. The erosion along cracks and joints had widened but not deepened. In July 1990, the stilling basin was in good condition with 2- to 8-in. (51- to 203-mm) depths of erosion observed along nearly the entire upstream floor.

Los Angeles River Low-Flow Channel

Background

The Los Angeles River is located in the Los Angeles District and flows through the city of Los Angles, CA, draining a 753-sq-mi (1,950-km^2) watershed. In the 1940's, the majority of the 50-mile (80-km) long river channel was constructed and paved with 12-in. (305-mm) thick, conventional, reinforced concrete. During the passing years, various features of the concrete channel had deteriorated or been damaged. The most significant damage occurred within a 12-mile (19-km) reach of the low-flow channel as the result of abrasion erosion caused by water-borne sediment. In some reaches, the erosion had progressed completely through the concrete slab. Minor abrasion erosion damage also occurred in the main channel invert.

Repair

The low-flow channel was repaired during 1983 through 1985 using silica-fume concrete (Holland and Gutschow 1987). The repair began in 1983 by removing the existing concrete in a 1/2-mile (0.8-km) reach containing the most severely eroded concrete and repaving with reinforced, silica-fume concrete. Subsequent rehabilitations of the channel were accomplished by either full-depth slab replacement or overlay on the existing concrete.

Full-depth repair included replacing existing concrete with 12-in. (305-mm) of reinforced concrete and overlaying the new slab with 6 in. (152 mm) of unreinforced, silica-fume concrete. Overlays in other areas were placed 4 in. (102 mm) or 6 in. (152-mm) thick using silica-fume concrete. Areas requiring patching were saw cut a minimum of 1 in. (25 mm) outside the perimeter of damage and concrete within was removed by chipping. Depths

of saw cuts were 3 in. (76 mm) for slabs and 2 in. (51 mm) for walls. All surfaces were wet sandblasted prior to placement of concrete.

The overlay concrete was specified to have 600 to 650 lb/cu yd (356 to 386 kg/cu m) of Type II portland cement, 70 to 90 lb/cu yd (41.5 to 53.4 kg/cu m) of silica fume, and a slump of 2 to 5 in. (51 to 127 mm). The concrete mixture proportions were modified throughout the rehabilitation. This resulted in a range of compressive strengths from 8,000 to 10,500 psi (55.2 to 72.4 MPa).

Concrete surfaces to be overlaid were prepared by wet sandblasting. A bond breaker curing compound conforming to the American Association of State Highway and Transportation Officials (AASHTO) designation M 148 (AASHTO 1986b), Type 2, white pigmented, was specified to be applied at rate of 1 gal/150 ft^2 (0.272 L/m^2) to the surfaces to be overlaid.

The total cost for the silica-fume concrete overlays was $\$28/yd^2$ ($\$33/m^2$) which included the $\$2/yd^2$ ($\$2/m^2$) cost for surface preparation (wet sandblasting). The estimated resurfaced area was 160,000 yd^2 (134,000 m^2). For comparison purposes, the total cost for fly-ash (Class F) concrete overlays used to rehabilitate the Los Angeles River Main Channel (1986-1988) was $\$21/yd^2$ ($\$25/m^2$). The estimated resurfaced area was 733,000 yd^2 (613,000 m^2).

Performance

Inspections of the channel have occurred during the construction of the last phases and subsequent to the last rehabilitation contract and the initial performance of the overlays was judged to be excellent. The next scheduled inspection is for fiscal year 1994.

Lowell Creek Dam Diversion Tunnel

Background

Lowell Creek Dam Project is located in the U.S. Army Engineer District, Alaska, near Seward, Alaska, and consists of a small, rock-filled dam across a canyon floor and a diversion tunnel through an adjoining mountain, with inlet and outlet transitions. Construction began in August 1939 and was completed in April 1945. The tunnel is just over 2,000 ft (610 m) in length at a 4-percent slope, of typical horseshoe section (Figure 22), and 10 ft (30.5 m) high at the center. All features are reinforced concrete, supplemented with steel armor rails along the invert to resist wear from extensive bedload during high flows.

Most of the steel armor rails, except for those along the very edges of the invert, had been lost or removed over time due to erosion of protecting

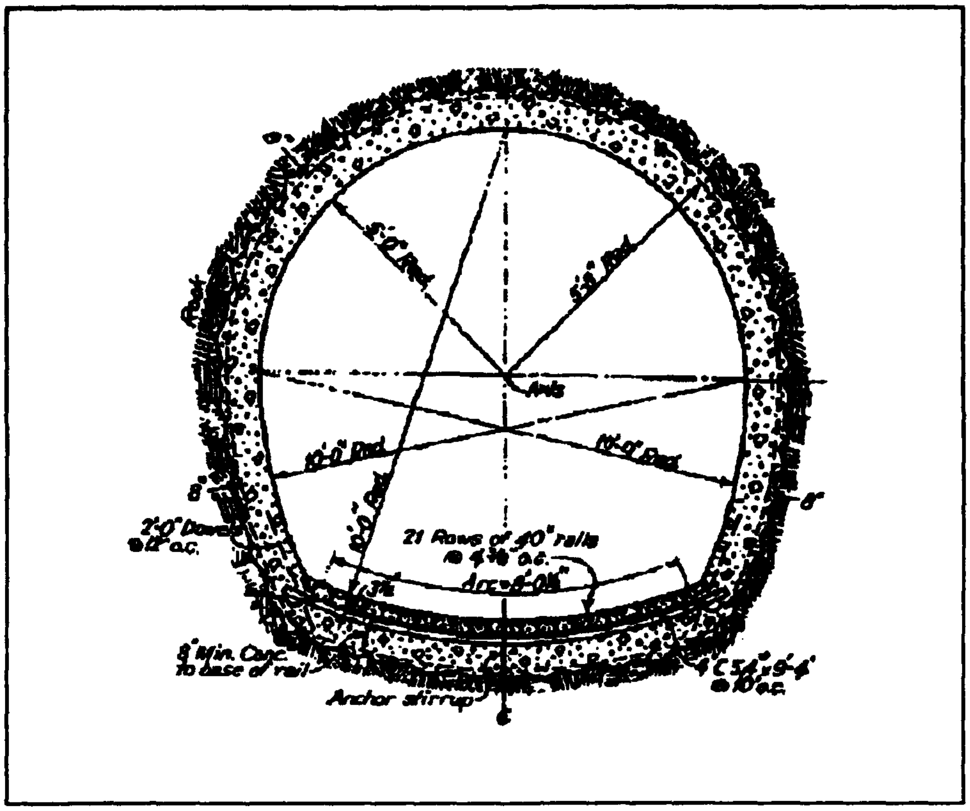

Figure 22. Sketch of tunnel floor overlay, Lowell Creek Dam Diversion Tunnel, 1991

concrete. In many places, the original, 8-in. (203-mm) thick concrete floor was completely scoured out.

A number of repairs have been accomplished since the 1960's including one in early 1988 in which the invert was repaired by filling the worst scour holes with conventional, 3,000 psi (20.7 MPa) concrete and overlaying a 150-ft (45.7-m) test section immediately upstream of the middle of the tunnel with 12 in. (305 mm) of nonreinforced, 8,000 psi (55.2 MPa), silica-fume concrete. Subsequent inspections showed little wear on this surface, even after significant flow with associated bedload. At the same time, damage reappeared in the other repair areas and increased appreciably where no repairs had been accomplished. Consequently, additional repairs were accomplished in 1991 using silica-fume concrete.

Repair

For the 1991 repair, holes in the invert were filled with a conventional, 3,000-psi (20.7-MPa) concrete and the tunnel and outlet overlaid with 9 in. (229 mm) of nonreinforced, 10,000-psi (69.0-MPa), silica-fume concrete between the edge rails (Figures 23 and 24). Surface was prepared using a high-pressure (6,000 to 10,000-psi (41.4 to 69.0-MPa)) water jet. Concrete

Figure 23. Placement of overlay, Lowell Creek Dam Diversion Tunnel, 1991

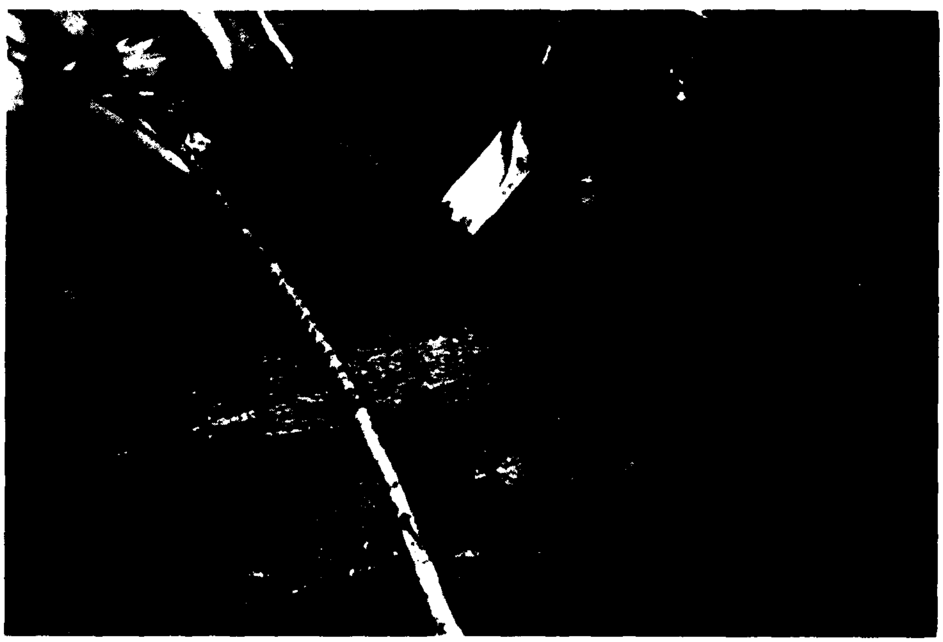

Figure 24. Construction joint, Lowell Creek Dam Diversion Tunnel, 1991

was batched at a central plant approximately 5 miles (8 km) from the project and a water-reducing admixture added at the job site. Concrete was transported by crane from transit mixers below the outlet transition to a trailer-mounted hopper above. Concrete was then transported by skid loader to points of placement within the tunnel. Placement in the outlet was by pump. All concrete was moist cured.

The concrete was specified to contain a Type I or Type II portland cement and to have a required average compressive strength of 11,500 psi (79.3 MPa), a maximum water-cement ratio by mass of 0.45, an air content between 2 and 4 percent, and a slump between 4 and 6 in. (102 and 152 mm). The concrete was batched using the following mixture proportions per cubic yard (cubic metre):

Portland cement	658 lb	(391 kg)
Silica fume	100 lb	(59.4 kg)
Coarse aggregate	1,763 lb	(1,046 kg)
Fine aggregate	1,270 lb	(754 kg)
Water	261 lb	(155 kg)
Air-entraining admixture	As required	
Water-reducing admixture, conventional	19.7 fl oz	(583 mL)
Water-reducing admixture, high-range	75.8 fl oz	(2,240 mL)

The conventional concrete was specified to have a slump between 4 and 6 in. (102 and 152 mm). It was also specified that calcium chloride not be used in the mixtures.

The total cost for the 1991 repairs was \$770/yd^2 (\$921/m^2). The estimated area overlaid was 916 yd^2 (766 m^2). A summary of cost are presented in Table 3.

Table 3
Repair cost, Lowell Creek Dam Diversion Tunnel, 1991

Item	Unit Cost \$/yd^2 (\$/m^2)	Cost \$
Mobilization and demobilization	169 (202)	155,000
Dewatering and tunnel weatherization	141 (169)	130,000
Debris removal and surface preparation	55 (66)	50,000
Filler concrete	48 (57)	44,000
Concrete liner	250 (299)	229,000
Modification for quantity underruns	8 (10)	7,300
Modification to include outlet transition	98 (117)	89,600
Total Cost	770 (921)	704,900

Performance

The tunnel invert was surveyed immediately following the 1991 repairs. In March 1992, a follow-up inspection and survey revealed a possible 1/8- to 1/4-in. (3- to 6-mm) wear along the center of the invert. It was noted that no heavy flows have been experienced since the repair.

6 Polymer-Modified Concrete Overlays

Apartment Balcony Floors: Guelph, Ontario, Canada

Background

The Central Region of the Ontario Ministry of Housing had the responsibility of maintaining an apartment complex in Guelph, Ontario, Canada. Over the years, deicing salts had been used to melt snow and ice accumulations on the concrete balcony floors. As a result, scaling and cracking developed as did the potential of structural damage due to corrosion of reinforcing steel.

Repair

The balcony floors of the apartment complex were repaired in 1991 and 1992 using a reinforced polymer-modified-mortar overlay known as Perma-Plate by Perma-Plate Systems, Inc. Surface preparation consisted of removing loose and delaminated concrete; removing surface contaminates such as paint and efflorescence by power brushing or sandblasting; and filling depressions with a conventional grout or mortar to provide a profile of 7/8 to 1 in. (22 to 25 mm) below final finish grade of overlay surface. Steel Tapcon screws were installed at 1-ft (0.305-m) spacings with welded-wire fabric (6 by 6 in. (152 by 152 mm) secured to screws. Sheets of a fine, mild, steel mesh, 5/8 by 48 by 86 in. (16 by 1,220 by 2,184 mm) were secured atop welded-wire fabric and to adjacent sheets.

The mortar was batched using portland cement, acrylic polymer, and natural sand. The portland cement was a Canadian Type 10 that complied with standard CAN3-A5-M77. The properties specified for the hardened mortar included 28-day strengths of 4,000-psi (27.6-MPa) compressive (ASTM C 39) (1988c) and 630-psi (4.3-MPa) tensile splitting (ASTM C 496) (1988f); 2,700,000-psi (18,600-MPa) modulus of elasticity at 28-day age (ASTM C 469 (1988e)); very low chloride-ion permeability (AASHTO T 277 (1986c)); freeze-thaw resistance to exceed durability factor of 60 percent at

300 cycles (ASTM C 666) (1988g); accelerated corrosion resistance of no deterioration after 100 hr (ASTM B 117) (1988a); and resistance to weathering of no deterioration after 100 hr (ASTM G 23) (1988i).

The mortar mixture proportions were specified as:

Material	Quantity
Cement	88 lb (40 kg)
Sand (dry)	265 lb (120 kg)
Polymer	3.09 gal (11.7 L)
Water	2.27 gal (8.6 L)

The sand grading limits were specified as:

Sieve Size	Percent Passing
4.75-mm (No. 4)	100
2.36-mm (No. 8)	85-97
1.18-mm (No. 16)	50-85
600-μm (No. 30)	25-60
300-μm (No. 50)	10-30
150-μm (No. 100)	2-10

Mortar was placed onto mesh sheets on top of the balcony floors and then vibrated thoroughly. Surface was screeded at a fixed level to provide a minimum cover of 3/16 in. (5 mm) above mesh. The vertical and underside faces of balcony floors were formed and mortar placed and vibrated. All placing and finishing was specified to be completed within about 20 min of batching. The mortar was wet cured for 48 hr and opened for use after 24 hr of dry curing. A sketch of the overlay is shown in Figure 25, and the completed repair is shown in Figure 26.

The unit cost for overlaying balcony floors was around \$105 (Canadian dollars)/yd^2 (\$126/m^2) of overlay. The total area resurfaced (50 balconies) was estimated to be 270 yd^2 (226 m^2).

Performance

The mortar overlay was accepted as complete in July 1992 via inspection by the Ontario Housing Commission with no deficiencies noted. Approximately 25 balconies (half the repair) have gone through one winter and have shown no signs of distress.

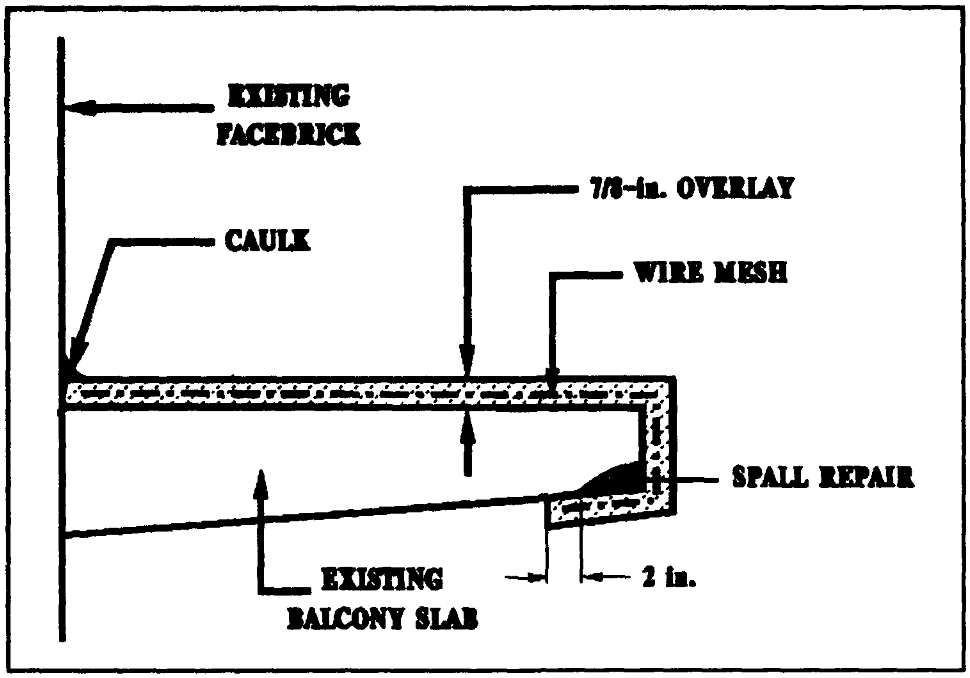

Figure 25. Sketch of overlay repair to apartment balconies, Guelph, Ontario, Canada, 1992

Figure 26. Completed repair of apartment balconies, Guelph, Ontario, Canada, 1992

Mahoning Creek Dam Spillway Bridge

Background

Mahoning Creek Dam is a Pittsburgh District structure located in Armstrong County, Pennsylvania, on Mahoning Creek 22.9 miles (36.8 km) upstream from the junction of the creek and the Allegheny River. The dam is a 926-ft (282-m) long, concrete gravity type structure that was completed in 1941 and includes a roadway across dam and spillway. The spillway portion of the dam consists of a reinforced-concrete roadway bridge, an ogee overflow section with five crest gates, three main and one smaller sluice outlets, and a stilling basin.

The dam was constructed using nonair-entrained concrete. As a result of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces developed moderate to severe spalling and scaling along with numerous random hairline cracks.

Repair

The spillway bridge deck was resurfaced in 1987 with a polymer-modified concrete. Surface preparation included the removal of 1-1/2 in. (38 mm) of concrete using a scabbler and patching local areas where removal was more extensive with a mortar to produce a uniform surface profile. The specifications required that the concrete substrate have an aggregate profile of $\pm$1/16 in. ($\pm$2 mm). Prepared surface was wetted prior to concrete placement.

Patching mortar consisted of one part cement and two parts clean, fine sand measured by volume with just enough water to produce a mixture of sufficient workability. Mortar was cured for a minimum of 24 hr.

The polymer-modified concrete was placed to a minimum thickness of 1-1/2 in. (38 mm). The approximate mixture proportions for the concrete was one unit of SikaTop 122 Repair Mortar to 42 lb (19.1 kg) of 9.5-mm (3/8-in.) coarse aggregate. The aggregate was required to be nonangular, clean, well graded, and saturated surface dry. Concrete was wet cured for a minimum of 24 hr.

After drying for a minimum of 24 hr, the surface was cleaned by light abrasive blasting and dust was removed by sweeping and compressed air. The concrete surface was then sealed with Fosroc Nitocote Dekguard.

The cost for the overlay, excluding removal and patching cost, was \$40/yd^2 (\$48/m^2). The cost for concrete removal was \$60/yd^2 (\$72/m^2) and for patching, \$126/cu ft (\$4,450/cu m). The cost of sealing the resurface was \$6/yd^2 (\$7/m^2). The estimated resurfaced area was 780 yd^2 (652 m^2).

Performance

The polymer-modified concrete overlay had numerous fine cracks.

Mississippi River Lock 3

Background

Mississippi River Lock 3 is located in the U.S. Army Engineer District, St. Paul, on the Mississippi River near Welch, MN. The project was completed in 1938 and includes a 600-ft (183-m) long by 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained. As a result of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces scaled and deteriorated.

Repair

In 1990 and 1991, portions of the top surfaces of the lock walls and guide walls were resurfaced or patched with latex-modified concrete. The bid specifications called for a styrene-butadiene latex; however, in the submittal phase, an acrylic latex (SikaLatex, manufactured by Sika Corporation) was approved as a substitute. The perimeter of damage was specified to be saw cut a minimum of 1-in (25-mm) deep, and the concrete within was removed to a depth of 1-1/2 in. (38 mm). Control joints were specified to be tooled in overlay above remaining cracks in surface of concrete after removal (Figure 27).

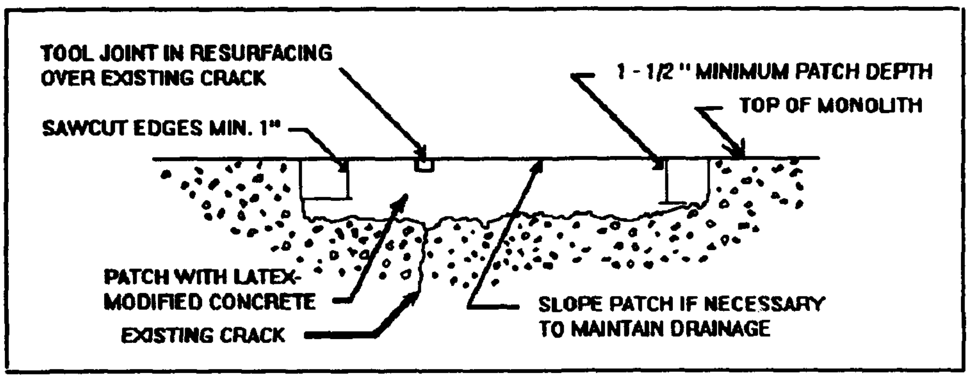

Figure 27. Typical surface repair, Mississippi River Lock 3, 1991

The concrete replacement cost was \$8/yd^2 (\$10/m^2) for 2-in.(51-mm) deep patch or overlay. The size of the individual resurfaced areas varied with the maximum being 53 yd^2 (44 m^2).

Performance

Hairline cracks along half of the repair edges and hairline cracks within the repair areas on about 10 percent of the repairs were observed during an inspection in October 1991.

Mississippi River Lock 22

Background

Mississippi River Lock and Dam 22 is located in the Rock Island District on the Mississippi River near Hannibal, MO. The project was completed in 1935 and includes a 600-ft (183-m) long and 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and contained crushed limestone coarse aggregate. As a result of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces scaled and deteriorated. Some nondurable aggregate particles were also present that increased the extent of deterioration.

Repair

In 1989, the top surface of the lock walls was resurfaced with a latex-modified concrete having a specified compressive strength of 4,000 psi (27.6 MPa). The concrete specifications required a Type I portland cement, 12.5-mm (1/2-in.) nominal maximum size aggregate, air content of 3 to 6 percent, and slump of 3 to 5 in. (76 to 127 mm). The latex emulsion used was Tylac 97-314 manufactured by Reichhold Chemicals, Inc. The concrete was batched using the following mixture proportions per cubic yard (cubic metre) of concrete:

Cement	658 lb	(391 kg)
Fine aggregate (crushed limestone)	1,645 lb	(976 kg)
Coarse aggregate (natural sand)	1,315 lb	(780 kg)
Latex emulsion	24.5 gal	(92.7 L)
Water	154 lb	(91.4 kg)

A cold planer (Figure 28) was used to remove 1-1/2 in. (38 mm) of concrete from the top surface of lock walls. Hand-held breakers were used to remove concrete in congested areas and in areas where steel reinforcement was expected. The concrete surface was cleaned by sandblasting and waterblasting. The surface was thoroughly wetted 1 hr prior to placement. The wet surface was covered with polyethylene to ensure the bond surface would remain wet and clean. The polyethylene was removed and a latex-modified grout was brushed into the surface immediately ahead of placement and consolidation of the latex-modified concrete. A broom finish was applied. Curing consisted of 24 hr of wet curing followed by 72 hr of dry curing. Wet burlap covered with polyethylene was used to effect the wet curing.

Figure 28. Cold planer used to remove concrete from the tops of lock walls, Mississippi River Lock 22, 1989

It was important that during the hot summer curing of the latex-modified concrete be implemented as soon as possible to prevent plastic shrinkage cracks from developing. However, for a number of placements, the burlap and polyethylene were placed on the finished concrete too soon and resulted in rough surface profile (Figure 29).

The concrete removal cost was $39/yd^2 ($47/m^2), and the latex-modified concrete cost was $50/yd^2 ($60/m^2). The estimate resurfaced area was 2,544 yd^2 (2,127 m^2).

Performance

Minor cracking was observed in the overlay during a visit to Mississippi River Locks 22 in August 1991. Cracking was most frequent at corners of blockouts (Figure 30). These cracks were random in direction and typically intersected other cracks that were random in direction. In general, the cracks appeared to be: (a) smaller in width and length than similarly located cracks seen at Mississippi River Locks 17, 18, and 20 where 4-in. (102-mm) deep low-slump concretes had been placed; and (b) slightly larger longer than similarly located cracks seen at Mississippi River Lock 21 where 4-in. (102-mm) deep, polypropylene-fiber-reinforced concrete had been placed.

Figure 29. Rough surface due to premature placement of polyethylene and burlap, Mississippi River Lock 22, 1989

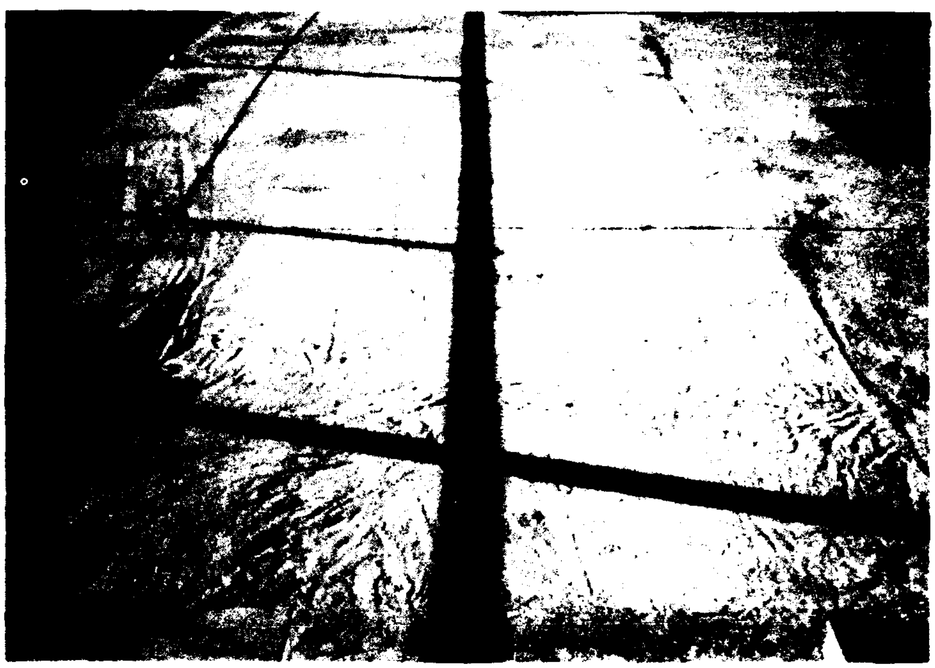

Figure 30. Cracking typically found at corners of blockouts, Mississippi River Lock 22, 1989

Reedy Point Bridge

Background

Reedy Point Bridge is located in the U.S. Army Engineer District, Philadelphia, near Delaware City, DE, and carries two lanes of State Route 9 over the Chesapeake and Delaware Canal. The bridge is a high-level, 8,432-ft (2,570-m) long truss and deck girder structure that was completed in 1969. The bridge structure consists of 67 spans; a 600-ft (183-m) suspended through truss main span with 300-ft (91.4-m) anchor spans at the north and south ends, each flanked by a 254-ft (77.4-m) deck truss span. The bridge deck was constructed on permanent metal deck forms and consisted of a 7-in. (178-mm) thick reinforced concrete slab having a 26-ft (7.92-m) width. The sidewalks on the east and west sides of the roadway are 1-1/2-ft (0.457-m) thick and 2-1/2-ft (0.762-m) wide with 8-1/2-in. (216-mm) high curbs.

As a result of frequent cycles of freeze-thaw exposure and the application of deicing salts, corrosion of the reinforcing and spalling of the concrete had occurred.

Repair

Reedy Point Bridge Deck was overlaid in 1985 with a latex-modified concrete having a specified 28-day compressive strength of 4,000 psi (27.6 MPa). Surface preparation included the removal of 1/4 in. (6 mm) of concrete and patching local areas where deterioration was more extensive. Deck surface was wetted prior to concrete placement.

The mixture design included Type I portland cement, concrete sand with a specific gravity of 2.61, Delaware Number 107 coarse aggregate having a specific gravity of 2.70, and a styrene-butadiene latex admixture ("DPS Modifier A" by Dow Chemical Company). Air content was specified to be 5 percent. The mixture proportions per cubic yard (cubic metre) were as follows:

Cement	658 lb (391 kg)
Fine aggregate	1,531 lb (909 kg)
Coarse aggregate	1,216 lb (722 kg)
Latex admixture (24.5 gal (92.7 L))	208 lb (123 kg)
Water (17.5 gal (66.2 L))	146 lb (866 kg)

The mortar portion of the concrete mixture was applied ahead of the placement of the overlay concrete. The overlay concrete was placed to a minimum thickness of 1-1/2 in. (38 mm). Transverse grooves were created in the overlay surface using a tine drag at time of finishing. Concrete was wet cured for a minimum of 24 hr using wet burlap cover with polyethylene and then dry cured for 72 hr.

The cost for the overlay excluding removal and patching cost was \$34/yd^2 (\$41/m^2). The cost for scarification of deck surface was \$4/yd^2 (\$5/m^2). The cost was \$126/yd^2 (\$151/m^2) for repairing 530/yd^2 (443/m^2) of bridge deck that required more extensive removal and patching. The estimated resurfaced area was 23,800 yd^2 (19,900 m^2).

Performance

During the September 1990 inspection of bridge deck overlay, hairline map cracking (Figure 31) was observed in all spans and medium map cracking in one span (Figure 32). There were numerous areas of hollow sounding concrete associated with the map cracking. Transverse cracking near joints (Figure 33) was also observed. The next inspection is scheduled in 1992.

Tygart Dam Roadway

Background

Tygart Dam is located in the Pittsburgh District on Tygart River about 23 miles (37 km) above the mouth of the river at Fairmont, West Virginia. The dam is a 1,921-ft (586-m) long, concrete gravity type structure that was completed in 1938 and includes a spillway and a roadway atop dam abutments. The spillway portion of dam consists of an uncontrolled ogee overflow section and eight main and two low-discharge sluice outlets.

The dam was constructed using nonair-entrained concrete. As a result of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces developed moderate to severe spalling and scaling along with numerous random hairline cracks.

Repair

The roadway on top of the dam was resurfaced in 1986 with polymer-modified concrete. Surface preparation included the removal of 1-1/2 in. (38 mm) of concrete using a scabbler and patching local areas where deterioration was more extensive with a mortar to produce a specified surface profile. The prepared roadway surface shown in Figure 34 was required to have an aggregate profile of $\pm 1/16$ in. (± 2 mm). The surface was wetted prior to placement of new concrete and covered with polyethylene to protect surface against contaminates leaked from equipment used (Figure 35).

Patching mortar consisted of one part cement and two parts fine sand by volume with just enough water to produce a mixture of sufficient workability. Mortar was cured for a minimum of 24 hr prior to placement of overlay concrete.

Figure 31. Typical hairline map cracking observed in overlay, Reedy Point Bridge, 1990

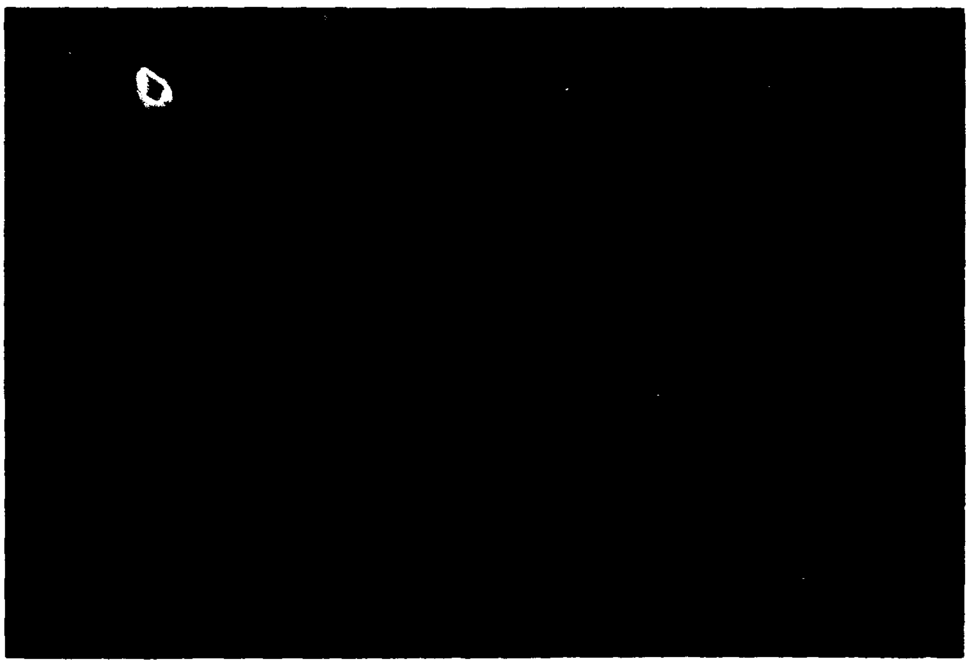

Figure 32. Medium map cracking observed in one span of bridge overlay, Reedy Point Bridge, 1990

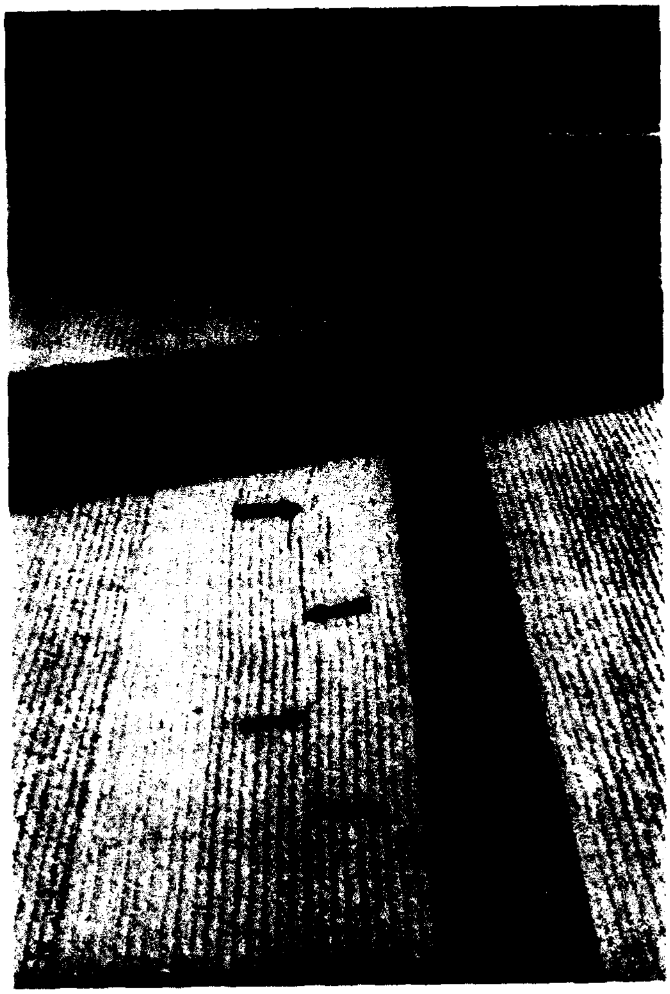

Figure 33. Transverse crack in overlay near joint, Reedv Point Bridge, 1990

Figure 34. Prepared surface, Tygart Dam Roadway, 1986

Figure 35. Polyethylene used to shield prepared bond surface from contaminants, Tygart Dam Roadway, 1986

The polymer-modified concrete was placed (Figure 36) to a minimum thickness of 1-1/2 in. (38 mm). The approximate mixture proportions for the concrete was one unit of SikaTop 122 Repair Mortar to 42 lb (19.1 kg) of 3/8-in. (10-mm) coarse aggregate. The aggregate was required to be nonangular, clean, well graded (conforming to AASHTO M 43, Size No. 8) (AASHTO 1986a) , and saturated surface dry. In one small area, the overlay was 1/8 to 1/4 in. (3 to 6 mm) thick. As a result, the polymer-modified concrete did not cover several spots and delaminated and spalled in surrounding area (Figure 37). The area was later bush hammered to specified depth and patched by the contractor. The concrete was wet cured for a minimum of 24 hr. In some areas, wet curing was applied too soon, leaving surface rough (Figure 38).

After drying for a minimum of 24 hr, the surface was cleaned by light abrasive blasting and dust removed by sweeping and compressed air. The concrete surface was then sealed with Fosroc Nitocote Dekguard.

The cost for the overlay, excluding concrete removal and patching, was around \$40/yd^2 (\$48/m^2). The cost for the concrete removal was \$16/yd^2 (\$19/m^2), and the cost for sealing the overlay surface was \$15/yd^2 (\$18/m^2). The resurfaced area was 1,215 yd^2 (1,016 m^2).

Performance

The overlay had numerous fine cracks.

Figure 36. Placement of polymer-modified concrete, Tygart Dam Roadway, 1986

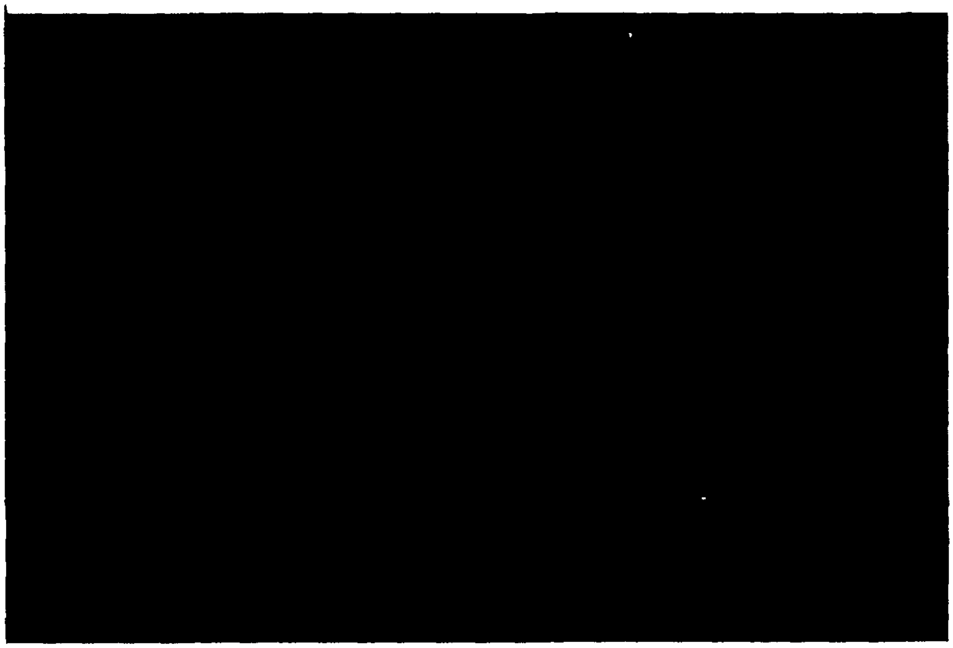

Figure 37. Spalling in area of inadequate placement depth, Tygart Dam Roadway, 1986

Figure 38. Rough surface due to premature placement of polyethylene and wet burlap, Tygart Dam Roadway, 1986

W. G. Huxtable Pumping Plant

Background

W. G. Huxtable Pumping Plant is a part of the Mississippi River Flood Control System and is located in the U.S. Army Engineer District, Memphis, near Marianna, AR. The plant is a reinforced-concrete structure that was completed in June 1977. It is actually three structures that are founded on soil cement foundation and are separated by joint material. The floodgate or center structure is 137 ft (41.8 m) wide (across channel) and 126 ft long (38.4 m). The two-pump or outside structures are 131 ft (39.9 m) wide by 126 ft (38.4 m) long.

The midheight slabs of the structures were subject to infiltration of water during pumping operations that caused water to stand about 3 in. (76 mm) above slabs. The slabs are 6 ft (1.8 m) thick and encase the lower portion of the intake pump bells in the two-pump structures. In the floodgate structure, the slab is 4 ft (1.2 m) thick. The three slabs are all constructed of reinforced concrete that was formed by conventional methods.

Repair

In January 1986, a contract was awarded to improve the seal between the pump bells and slab and to waterproof the slab. Approximately 2,100 yd^2 (1,760 m^2) of slab area was overlaid with an acrylic-modified concrete under this contract. The total cost for contract work was $97/$yd^2$ ($116/$m^2$).

Shotblasting was used to prepare the floor surface. An acrylic resin was specified as a primer coat at the rate of 1 gal/100 ft^2 (0.407 L/m^2). The mixture proportions for the overlay concrete were specified as follows:

Cement	20 lb	(9.1 kg)
Sand	45 lb	(20.4 kg)
1/4-in. (6-mm) gravel	15 lb	(6.8 kg)
Acrylic resin cut 30% by water	1 gal	(3.78 L)

The overlay was placed to its original elevation. The thickness of the overlay was approximately 1/4 in. (6 mm). A magnesium float was specified for use in finishing the surface, and the surface was required to have a "no skid" finish.

Performance

An inspection of the pumping plant in October 1990 revealed no problems with the repair. Minor leakage has been observed since the repair during operation of pumps. However, a determination has not been made for the source of leakage.

7 Fiber-Reinforced Concrete Overlays

Kinzua Dam Stilling Basin

Background

Kinzua Dam is located in the U.S. Army Engineer District, Pittsburgh, on the Allegheny River approximately 198 miles (319 km) above the mouth of the river at Pittsburgh, PA. The project was completed in 1965 and includes a concrete gravity spillway and stilling basin. The spillway contains four tainter-gate sections and an outlet works consisting of six low-level sluices through the spillway. The stilling basin is 204 ft (62.2 m) wide and 178 ft (54.3 m) long and contains nine 8-ft (2.4-m) high by 10-ft (3.0-m) wide by 18-ft 8-in. (5.7-m) long baffle blocks. The stilling basin floor was constructed a minimum of 5 ft (1.5 m) thick using concrete having a 6-in. (152-mm) nominal maximum size aggregate and a compressive strength of 3,000 psi (20.7 MPa) at 28-day age.

In 1969, abrasion erosion damage was observed in the stilling basin by scuba divers (McDonald 1980). Most of the damage was located at the contraction joints and corners of baffle blocks. Scour holes up to 42 in. (1,070 mm) deep and exposed reinforcing were also observed. By 1973, the erosion was present over most of the stilling basin floor.

Repair

A steel-fiber-reinforced concrete overlay was used to repair and raise the stilling basin floor 1 ft in elevation (McDonald 1980). Half of the repair was performed in 1973 and the other half in 1974. In preparation for the overlay, loose, weak, and deteriorated concretes were removed by chipping, and the surface was prepared by wet sandblasting or water jet blasting. The deeper holes were partially filled with conventional concrete having a compressive strength of 3,000 psi (20.7 MPa). A high-modulus epoxy bonding agent was placed just ahead of the fiber-concrete overlay.

The fiber-reinforced concrete was batched using a coarse to fine aggregate ratio of 1.5. The coarse aggregate was a crushed limestone. Each batch

contained 200 lb/cu yd (119 kg/cu m) of thin, 1-in. (25-mm) long, flat steel fibers. The fiber-reinforced concrete had a 6,000-psi (41.4-MPa) compressive strength and a 1,100-psi (7.6-MPa) flexural strength at 28-day age.

The concrete was batched 6 cu yd (4.6 cu m) at a time and transported to job site in a 10-cu-yd (7.6-cu-m) transit mixer. The fine and coarse aggregates were placed in mixer and approximately 70 percent of mixing water added. The steel fibers were then added using a high-speed conveyor at a nearly continuous rate. The cement was charged into the mixer and remaining water added prior to transporting the batch to the job site. The above order of batching eliminated or reduced occurrences of the balls of fibers in the batched concrete.

The cost for overlaying the stilling basin floor excluding costs for mobilization, dewatering, removal, and filling holes was \$143/yd^2 (\$171/m^2). The cost for removal was \$4/yd^2 (\$5/m^2). The total cost of repair was \$499/yd^2 (\$597/m^2). An estimated 3,440 yd^2 (2,880 m^2) of floor was overlaid.

Performance

An underwater inspection by divers in November of 1974 found minor abrasion erosion damage to some of the baffles and surrounding floor area. An estimated 45 cu yd (34 cu m) of debris was removed from stilling basin. In April 1975, several floor areas upstream of the baffles were eroded 5 to 17 in. (127 to 432 mm) deep exposing reinforcing. Trenches 4 to 12 in. (102 to 305 mm) deep were eroded around some of the baffles. Another 45 cu yd (34 cu m) of debris was removed from the stilling basin. An underwater inspection by divers in June 1977 indicated that erosion had reached a maximum depth of 36 in. (914 mm).

It was concluded though demonstrations that debris was being brought into the stilling basin from areas downstream. The movement of debris was attributed to nonsymmetrical discharges from the outlet works. The unbalanced flows increased eddy and return currents and were of sufficient velocities to transport riprap and aggregate from downstream into the basin. The operation of the sluice gates were revised to provide symmetrical discharges to reduce the potential for eddy and return currents.

After about 10 years of erosion damage, the stilling basin was again in need of repair. The Pittsburgh District selected a silica-fume concrete overlay to replace the steel-fiber-reinforced concrete based on results from a WES study to find the highest abrasion resistant concrete mixture for the repair of Kinzua Dam stilling basin (Holland 1983). The silica-fume overlay was placed in 1983 and is performing excellently with only minor abrasion erosion observed, even though significant cracking had occurred in the bonded overlay.

LaGrange Lock

Background

LaGrange Lock is located on the Illinois Waterways near Beardstown, IL. The project was completed in 1939 and includes a 600-ft (183-m) long by 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and contained a natural gravel coarse aggregate that was predominantly limestone and dolomite. As a result of frequent cycles of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and deteriorated. Some nondurable aggregates were also present that increased the extent of deterioration.

Repair

In 1987, the top surfaces of the lock walls were resurfaced with concrete containing polypropylene fibers. The fiber-reinforced concrete was an approved value engineering modification and was used in lieu of latex-modified concrete. The polypropylene fibers were Forta A-10.

After removal of 2 in. (51 mm) of concrete, the surface was cleaned by sandblasting and waterblasting. Grout consisting of portland cement, sand, and water was scrubbed into the surface prior to the overlay placement. The fiber-reinforced concrete mixture was stiff and difficult to finish.

The cost for concrete removal was \$55/yd^2 (\$66/m^2), and the cost for the overlay was \$37/yd^2 (\$44/m^2). The estimated overlay area was 1,060 yd^2 (886 m^2).

Performance

Minor cracking was observed in the overlay during a periodic inspection conducted in August 1991. Cracking was most frequent at corners of blockouts. These cracks were random in direction and typically intersected other cracks that were random in direction. Fibers were discernible in the overlay surface.

Mississippi River Lock 21

Background

Mississippi River Locks and Dam 21 is located in the Rock Island District on the Mississippi River near Quincy, IL. The project was completed in 1935 and includes a 600-ft (183-m) long and 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and

contained a crushed limestone coarse aggregate. As a result of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces scaled and deteriorated. Some nondurable aggregates were also present that increased the extent of deterioration.

Repair

In 1987, the tops of lock walls were resurfaced with concrete containing polypropylene fibers. The fiber-reinforced concrete was an approved value-engineering modification and was used in lieu of latex-modified concrete.

A Gehl CP 400 cold planer (similar to the one shown in Figure 28) was used to remove 2 in. (51 mm) of concrete from the top surface of lock walls. Hand-held breakers were used to remove concrete in congested areas and in areas where steel reinforcing was expected. The concrete surface was cleaned by wet sandblasting and waterblasting.

The cost for concrete removal was \$36/yd^2 (\$43/m^2) and for the overlay was \$24/yd^2 (\$29/m^2). The estimated overlay area was 882 yd^2 (737 m^2).

Performance

Minor cracking was observed in the overlay during an August 1991 visit to the project. Cracking was most frequent at corners of blockouts. These cracks were random in direction and typically intersected other cracks that were random in direction. In general, the cracks observed during an August, 1991 visit appeared to be smaller in width and length than similarly located cracks seen at Mississippi River Lock Numbers 17, 18, and 20 where low-slump concretes were placed and slightly smaller than similarly located cracks at Mississippi Lock Number 22 where latex-modified concrete was placed.

Peoria Lock

Background

Peoria Lock is located on the Illinois Waterways near Peoria, IL. The project was completed in 1939 and includes a 600-ft (183-m) long by 110-ft (33.5-m) wide navigation lock. The concrete used to construct the lock was not air-entrained and contained a natural gravel coarse aggregate that was predominantly limestone and dolomite. As a result of exposure to freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces scaled and deteriorated. Some nondurable aggregate particles were also present that increased the extent of deterioration.

Repair

In 1987, the top surfaces of the lock walls were resurfaced with concrete containing polypropylene fibers. The fiber-reinforced concrete was an approved value engineering modification and was used in lieu of latex-modified concrete. The polypropylene fibers were Forta A-10.

A high-pressure water jet was used to remove 2 in. (51 mm) of concrete from the top surface of lock walls. Grout consisting of sand and water was scrubbed into the surface prior to the overlay placement. The fiber-reinforced concrete mixture was stiff and difficult to finish.

The cost for concrete removal was $\$120/yd^2$ ($\$144/m^2$), and the cost for the overlay was $\$32/yd^2$ ($\$38/m^2$). The estimated overlay area was 880 yd^2 (736 m^2).

Performance

Minor cracking was observed in the overlay during a periodic inspection conducted in August 1991. Cracking was most frequent at corners of blockouts. These cracks were random in direction and typically intersected other cracks that were random in direction. Fibers were discernible in the overlay surface.

8 Unbonded Concrete Overlays

Dashields Locks

Background

Dashields Locks are located on the Ohio River 13.3 miles (21.4 km) downstream of its point of origin at Pittsburgh, PA. The project was completed in 1928 and includes two parallel navigation locks, one 600 ft (183 m) long by 110 ft (33.5 m) wide and the other 360 ft (110 m) long by 56 ft (17.1 m) wide. The concrete used to construct the locks was not air-entrained. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and weathered (Figure 39).

Figure 39. Scaled and deteriorated surface on the top of a lock wall, Dashields Locks, 1985

A portion of the top of all three lock walls had been overlaid with a bonded, fly-ash concrete during 1988. The overlay was part of the overall rehabilitation of the locks and dam that began in 1987. Random cracking was observed at 5- to 10-ft (1.52- to 3.05-m) intervals in the overlay (U.S. Army Engineer Division, Ohio River 1989). A study to minimize the cracking problem was funded by the Ohio River Division and performed by the Waterways Experiment Station (WES). Recommendations from the study to minimize cracking in the overlays (Hammons, Garner, and Smith 1989) included: a) mixture proportions be selected to limit shrinkage, b) moist curing be used, c) bond breaker be employed, and d) thickness of the overlay be increased.

Repair

As a result of the WES study, the following changes were incorporated into the 1989 overlay work (U.S. Army Engineer Division, Ohio River 1989):

a. The concrete was reproportioned to reduce its shrinkage characteristic. The approximate mixture proportions per cubic yard (cubic meter) were as follows:

Material	Concrete	
	1988	1989
Cement	550 lb (326 kg)	480 lb (285 kg)
Fly ash	80 lb (47.5 kg)	120 lb (71.2 kg)
Fine aggregate	1,207 lb (716 kg)	1,243 lb (738 kg)
Coarse aggregate	1,730 lb (1,027 kg)	1,730 lb (1,027 kg)
Water	271 lb (161 kg)	270 lb (160 kg)
Air-entraining admixture	16.5 fl oz (488 mL)	9.6 fl oz (284 mL)
Water-reducing admixture	104.5 fl oz (3,090 mL)	86.4 fl oz (2,550 mL)

b. In most areas, a bond breaker consisting of two layers of 15-lb (8.9-kg), ASTM D 226, Type I (ASTM 1988h), asphalt-saturated felt was placed on top of the in situ concrete prior to overlaying (Figure 40).

c. A total of 233 ft (71.0 m) of dummy joints were employed to control random cracking (Figure 41). The bond breaker was not applied at mooring post locations as delineated by control joints shown. The joints were formed using metal strips to produce 1/4-in. (6-mm) wide grooves.

Figure 40. Bond breaker being installed, Dashields Locks, 1988

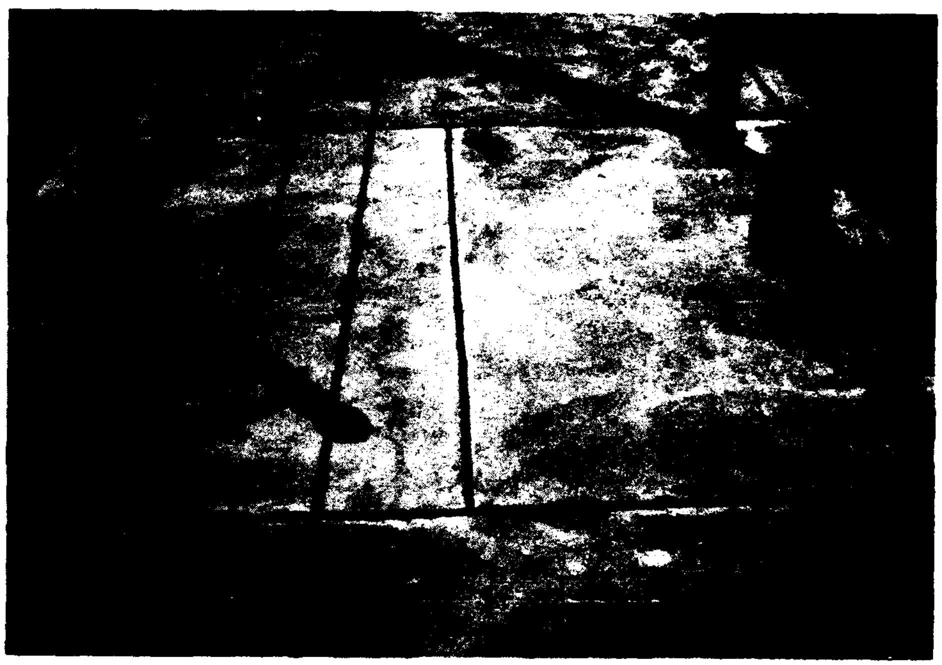

Figure 41. Dummy joints employed to control random cracking, Dashields Locks, 1989

The metal inserts were favored by field personnel over saw cutting. Details of dummy joints are illustrated in Figure 42. Note that the tooled V-joint is over exaggerated in this detail.

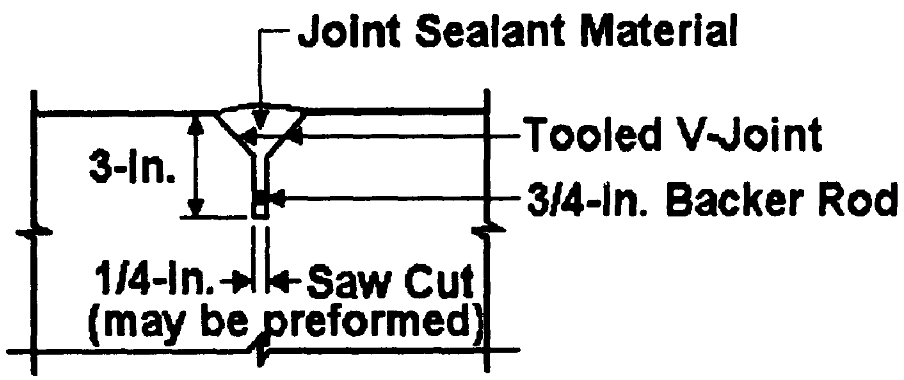

Figure 42. Details of dummy joint, Dashields Locks, 1989

d. Concrete was wet cured using burlene, a burlap and plastic sheet composite material, in lieu of burlap cover with polyethylene.

The top surface of lock walls was overlaid with an unbonded, 12-in. (305-mm) thick, fly-ash concrete. The concrete specifications required a 37.5-mm (1-1/2-in.) nominal maximum size aggregate, air content of 4 to 7 percent, a maximum water-cement ratio by mass of 0.45, a slump of 1 to 4 in. (25 to 102 mm), and a 28-day compressive strength of 4,000 psi (27.6 MPa). Surfaces were prepared by removing loosely bonded concrete with a shovel and broom. A nonshrink grout was used as a leveling course in areas where loose concrete was removed or concrete was spalled.

The reinforcement consisted of a mat of number 5 steel reinforcing bars on 12-in. (305-mm) centers in both the longitudinal and transverse directions. Number 6 (19-mm) bar dowels were installed on 24-in. (610-mm) centers along a line 1 ft (0.305-m) back from vertical face at both land and river edges of wall. Dowels were embedded a minimum of 15 in. (381 mm).

The cost for overlaying the tops of lock walls was $\$190/yd^2$ ($\$227/m^2$) which was a $\$20/yd^2$ ($\$24/m^2$) increase over the cost for the bonded overlay that was earlier employed. These unit costs excluded cost for concrete removal and reinforcement. The cost for removal was considered minimal due to the limited removal required. The total quantity for the unbonded overlay was 2,870 yd^2 (2,400 m^2).

Performance

The implementation of several of the changes recommended in the WES study appeared to have been beneficial since there was a reduction in the occurrence of cracks in the 1989 overlays from the occurrence observed in the 1988 overlays (U.S. Army Engineer Division, Ohio River 1989). The dummy joints appeared to control the location of random cracking as was evident in one area where a joint was terminated and crack continued (Figure 43). Overall, the concrete was in excellent condition during a periodic inspection in July 1991.

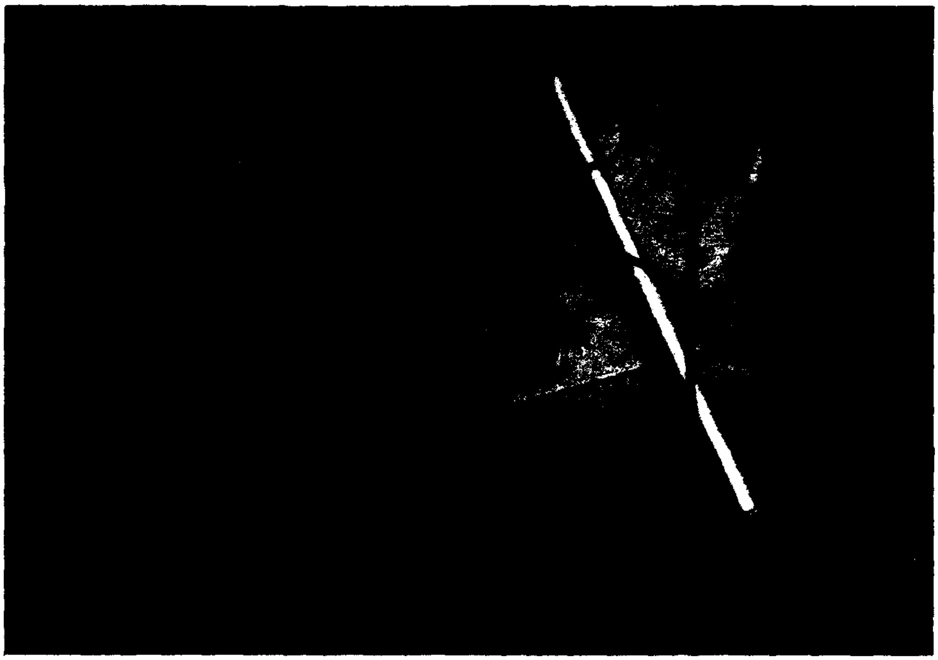

Figure 43. Crack continues from end of dummy joint, Dashields Locks, 1989

Interstate 96 at Portland, Michigan (Simonsen and Price 1989)

Background

Interstate 96 (I 96) was constructed in 1958 and 59 and generally consisted of pairs of two, 12-ft (3.7-m) wide lanes in which pavement thickness was 9 in. (229 mm), where conventionally reinforced concrete pavement was placed, and 8 in. (203 mm), where continuously reinforced concrete pavement was placed. Dowels, 1-1/4 in. (32 mm) in diameter and 18 in. (457 mm) long, were located on 12-in. (305-mm) centers across joints to provide load transfer. The joints were formed 1/2 in. (13 mm) wide and 2 in. (51 mm) deep and were sealed with hot-poured sealant. Pavement was constructed on a

clay grade with a 10-in. (254-mm) deep sand subbase and a 4-in. (102-mm) deep densely graded aggregate base. The sand base extended to the ditch foreslopes to provide drainage of subbase. The average daily traffic volume was 8,850 with 22 percent commercial. In 1984, the continuously reinforced concrete had been saw cut into 100-ft (30.5-m) slabs to reduce the movement at previously placed repairs and fractures in steel reinforcing. The conventionally reinforced concrete portions of roadway were in reasonably good condition prior to overlaying.

Repair

In 1984, a portion of I 96 at Portland, MI, was overlaid with a 7-in. (178-mm) thick, unbonded concrete. A 3/4-in. (19-mm) thick, sand-asphalt mixture (Michigan Department of Transportation Bituminous Mixture No. 1100T, 35A) was used as the bond breaker. The concrete specifications required a 28-day compressive strength of 3,500 psi (24.1 MPa), 6A gravel, and wire mesh weighing 6.3 lb/yd^2 (3.4 kg/m^2). Average of test results for 28-day compressive strengths was 5,970 psi (41.2 MPa).

Joint specifications required 18-in. (457-mm) long, 1-1/4-in. (32-mm) diameter, epoxy coated dowels on 12-in. (305-mm) centers across transverse joints and 24-in. (610-mm) long, No. 5, epoxy coated, deformed tie bars across longitudinal roadway center-line joint between lanes and across roadway-shoulder joints. Tie bars were spaced on 30-3/4-in. (781-mm) centers along roadway center line and on 54-2/3-in. (1,389-mm) centers along roadway-shoulder joints.

Transverse joints were sawn to create 9/16-in. (14-mm) wide by 2-1/8-in. (54-mm) deep grooves, and longitudinal joints were sawn to create 1/4-in. (6-mm) wide by 1-in. (25-mm) deep grooves. Transverse joints were located at least 3 ft (0.9 m) away from any joint or crack in the existing pavement. The maximum joint spacing for transverse joints was 100 ft (30.5 m). Transverse joints were sealed with preformed neoprene seal. Longitudinal joints were sealed with a hot-poured sealant.

Performance

The 1984 overlays on I 96 and U.S. 23 were evaluated as part study for the Michigan Department of Transportation (Simonsen and Price 1989) in which 7-in. (178-mm) thick, unbonded overlays were being evaluated as an alternative to repaving with 10-in. (254-mm) thick recycled concrete. Based on visual inspections, measurements, examination of cores, and load test, the overall performance was deemed to be satisfactory. Cracking observed in the overlays was more frequently located near or at cracks or joints in the original pavement beneath. The overlay pavements showed a 29-percent increase in load transfer efficiency at joints and a 25-percent decrease in midspan deflection when compared to adjacent 9-in. (229-mm) recycled pavement. The recycled concrete pavements were placed on a 4-in. (102-mm) deep,

open-graded drainage base. There were large variations in movements of the joints indicating the bond breaker does not allow fully independent movement of the overlay from the underlining concrete pavement. It was recommended that consideration be given to improve the effectiveness of the debonding layer.

A favorable life cycle cost for the overlay repair over placing recycled concrete pavement was indicated by the field and laboratory data. It was estimated that a savings of $35,000/mile ($21,800/km) of two-lane pavement would result from using the overlay repair in lieu of placing recycled concrete pavement.

Loyalhanna Dam Roadway

Background

Loyalhanna Dam is located in the Pittsburgh District on Loyalhanna Creek 4.5 miles (7.2 km) above its junction with the Conemaugh River at Saltsburg, PA. The dam is a 760-ft (232-m) long, concrete gravity type structure that was completed in 1942 and includes a roadway across dam. The spillway portion of dam consists of a reinforced-concrete roadway bridge, an ogee overflow section with five crest gates and four sluice outlets, and stilling basin.

The dam was constructed using nonair-entrained concrete. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had developed moderate to severe spalling and scaling along with numerous random hairline cracks in dam roadway and bridge surfaces.

Repair

A portion of the dam roadway was overlaid with an unbonded concrete in 1987 containing welded-wire fabric (6 in. by 6 in. (152 mm by 152 mm)). Surface preparation included the removal of loose and deteriorated concrete and the patching of spalled areas with a mortar mixture.

Patching mortar consisted of one part cement and two parts fine sand by volume with just enough water to produce a mixture of sufficient workability. Mortar was cured for a minimum of 24 hr prior to application of bond breaker.

A 1/16-in. (2-mm) thick, asphalt modified urethane membrane, similar and equal to Hydrocide Liquid Membrane No. 5000 (manufactured by Sonneborn Building Products), was specified as a bond breaker. The membrane was to cure for 24 hr prior to placing the concrete.

The concrete was placed to a minimum thickness of 4 in. (102 mm). The specifications for the concrete included a 28-day compressive strength of 3,000 psi (20.7 MPa), maximum nominal size coarse aggregate of 37.5 mm (1-1/2 in.), maximum water-cement ratio by mass of 0.60, air content of 4.5 to 7.5 percent, and a slump of 2 to 5 in. (51 to 127 mm).

After curing, the surface was cleaned by light abrasive blasting and dust was removed by sweeping and compressed air. The surface was then sealed with Nitocote Dekguard (manufactured by Fosroc, Inc.).

The cost for overlay excluding removal and patching was \$180/yd^2 (\$215/m^2). The cost for patching was \$700 per cubic yard, and for sealing the overlay surface, the cost was \$15/yd^2 (\$18/m^2). The overlay area was 205 yd^2 (171 m^2).

Performance

The unbonded concrete overlay was in very good condition with only occasional minor cracking.

Mahoning Creek Dam Roadway

Background

Mahoning Creek Dam is a Pittsburgh District structure located in Armstrong County, Pennsylvania, on Mahoning Creek 22.9 miles (36.8 km) upstream from the junction of the creek and the Allegheny River. The dam is a 926-ft (282-m) long concrete gravity type structure that was completed in 1941 and includes a roadway across the dam. The spillway portion of dam consists of a reinforced-concrete roadway bridge, an ogee overflow section with five crest gates and three main and one smaller sluice outlets, and stilling basin.

The dam was constructed using nonair-entrained concrete. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had developed moderate to severe spalling and scaling along with numerous random hairline cracks in dam roadway and bridge deck surfaces.

Repair

The roadway surface atop the dam abutments was overlaid in 1987 with an unbonded concrete containing welded-wire fabric (6 in. by 6 in. (152 mm by 152 mm)). Surface preparation included the removal of loose and deteriorated concrete and the patching of spalled areas with a mortar.

Patching mortar consisted of one part cement and two parts fine sand by volume with just enough water to produce a mixture of sufficient workability. Mortar was cured for a minimum of 24 hr prior to application of bond breaker.

A 1/16-in. (2-mm) thick, asphalt-modified urethane membrane was used as a bond breaker (Figure 44). The membrane was cured for 24 hr prior to placing the concrete.

Figure 44. Bond breaker in place for unbonded overlay, Mahoning Creek Dam Roadway, 1987

The concrete was pumped from end of dam abutment to placement location and placed to a minimum thickness of 4 in. (102 mm) (Figure 45). The specifications for the concrete included a 28-day compressive strength of 3,000 psi (20.7 MPa), nominal maximum size coarse aggregate of 37.5 mm (1-1/2 in.), maximum water-cement ratio by mass of 0.60, air content of 4.5 to 7.5 percent, and a slump of 2 to 5 in. (51 to 127 mm).

Dummy joints to control random cracking (Figure 46) were saw cut 1-1/4 in. (32 mm) deep and 1/4 in. (6 mm) wide. After curing, the overlay surface was cleaned by light abrasive blasting and dust was removed by sweeping and compressed air. The surface was then sealed with Nitocote Dekguard (manufactured by Fosroc, Inc.).

The cost for overlay excluding removal and patching was approximately \$29/yd^2 (\$35/m^2). The cost for patching repairs was \$126/cu ft (\$4,450/m^3)

Figure 45. Placement of concrete of unbonded overlay, Mahoning Creek Dam Roadway, 1987

Figure 46. Dummy joints used to control random cracking, Mahoning Creek Dam Roadway, 1987

and for sealing overlay surface, \$6/yd^2 (\$7/m^2). The overlay area was 795 yd^2 (665 m^2).

Performance

The unbonded overlay is in very good condition with only occasional fine cracks.

Montgomery Locks

Background

Montgomery Locks are located in the Pittsburgh District on the Ohio River 31.7 miles (51.0 km) downstream of the river's point of origin at Pittsburgh, PA. The project was completed in 1936 and includes two parallel navigation locks, one 600 ft (183 m) long by 110 ft (33.5 m) wide and the other 360 ft (110 m) long by 56 ft (17.1 m) wide. The concrete used to construct the locks was not air-entrained. As a result of exposure to frequent cycles of freezing and thawing in a critically water saturated condition, the nonair-entrained concrete surfaces had scaled and weathered.

Repair

A fiberglass fabric was employed as a bond breaker for the concrete overlay on the tops of river wall monoliths R-10 and R-24 during the 1985-1989 rehabilitation of the locks and dam. At the time, the tops of land, middle, and river lock walls were all being overlaid with a 12-in. (305-mm) thick, Type I, portland-cement concrete. The concrete specifications required a 37.5-mm (1-1/2-in.) nominal maximum size aggregate, an air content of 3.5 to 6.5 percent, a 2- to 5-in. (51- to 127-mm) slump, and a 28-day compressive strength of 3,000 psi (20.7 MPa).

The reinforcement consisted of a mat of No. 5 (16-mm) bars on 12-in. (305-mm) centers in both the longitudinal and transverse directions. A No. 5 (16-mm), 4-ft (1.2-m) long reinforcing bar was placed diagonally at all reentrant corners of blockouts to limit cracking of the overlay in this area. No. 6 (19-mm) bar dowels were installed on 24-in. (610-mm) centers in both the longitudinal and transverse directions. Dowel were embedded a minimum of 24 in. (610 mm).

Performance

During a periodic inspection of Montgomery Locks and Dam in 1989, the extent of cracks observed in the unbonded concrete overlay were significantly less than those observed in other monoliths where overlay was bonded.

U.S. Highway 23 near Dundee, Michigan (Simonsen and Price 1989)

Background

U.S. Highway 23 was constructed in 1959 to 1961 and consists of two, 12-ft (3.7-m) wide, 9-in. (229-mm) thick, conventionally reinforced concrete lanes with 99-ft (30.2-m) joint spacings. Dowels, 1-1/4-in. (32-mm) diameter and 18-in. (457-mm) long, were located on 12-in. (305-mm) centers across joints to provide load transfer. The joints were formed 1/2 in. (13 mm) wide and 2 in. (51 mm) deep and were sealed with hot-poured sealant. Pavement was constructed on a clay grade with a 10-in. (254-mm)-deep sand subbase and a 4-in. (102-mm) deep densely graded aggregate base. The sand base extended to ditch foreslopes to provide drainage of subbase. The average daily traffic volume was 11,750 with 16 percent commercial. By 1984, the highway was in relatively poor condition with many of the joints and interior slab cracks heavily patched with bituminous cold patch material.

Repair

In 1984, a portion of U.S. Highway 23 near Dundee, Michigan was overlaid with a 7-in. (178-mm) thick, unbonded, fly-ash concrete. A 3/4-in. (19-mm) thick, sand-asphalt mixture (Michigan Department of Transportation Bituminous Mixture No. 1100T, 35A) was used as the bond breaker. The concrete specifications required a 28-day compressive strength of 3,500 psi (24.1 MPa) coarse aggregate consisting of a 6A crushed limestone and wire mesh weighing 6.3 lb/yd^2 (3.4 kg/m^2). Average of test results for 28-day compressive strengths was 5,820 psi (40.1 MPa). Mixture proportions per cubic yard included 479 lb (284 kg) of portland cement, 72 lb (42.7 kg) of fly ash, and a water reducer.

Joint specifications required 18-in. (457-mm) long, 1-1/4-in. (32-mm) diameter, epoxy coated dowels on 12-in. (305-mm) centers across transverse joints and 24-in. (610-mm) long, No. 5, epoxy coated, deformed tie bars across longitudinal roadway center-line joint between lanes and across roadway-shoulder joints. Tie bars were spaced on 30-3/4-in. (781-mm) centers along roadway center line and 54-2/3-in. (1,389-mm) centers along roadway-shoulder joints.

Transverse joints were sawn to create 9/16-in. (14-mm) wide by 2-1/8-in. (54-mm) deep grooves, and longitudinal joints were sawn to create 1/4-in. (6-mm) wide by 1-in. (25-mm) deep grooves. Transverse joints were located at least 3 ft (0.9 m) away from any joint or crack in the existing pavement. The maximum joint spacing for transverse joints was 41 ft (12.5 m). Transverse joints were sealed with preformed neoprene seal. Longitudinal joints were sealed with a hot-poured sealant.

Performance

The 1984 overlays on I 96 and U.S. 23 were evaluated as part of a study for the Michigan Department of Transportation (Simonsen and Price 1989) in which 7-in. (178-mm) thick unbonded overlays were being evaluated as an alternative to repaving with 10-in. (254-mm) thick recycled concrete. Based on visual inspections, measurements, examination of cores, and load tests, the overall performance was deemed to be satisfactory. Cracking observed in the overlays was more frequently located near or at cracks or joints in the original pavement beneath. The overlay pavements showed a 29-percent increase in load transfer efficiency at joints and a 25-percent decrease in midspan deflection when compared to adjacent 9-in. (229-mm) recycled pavement. The recycled concrete pavements were placed on a 4-in. (102-mm) deep, open-graded drainage base. There were large variations in movements of the joints indicating the bond breaker does not allow fully independent movement of the overlay from the underlining concrete pavement. It was recommended that consideration be given to improve the effectiveness of the debonding layer.

A favorable life cycle cost for the overlay repair over placing recycled concrete pavement was indicated by the field and laboratory data. It was estimated that a savings of $35,000/mile ($21,800/km) of two-lane pavement would result from using the overlay repair in lieu of placing recycled concrete pavement.

9 Discussion

The case histories presented were typically for overlays completed within the last 10 years and located at a Corps of Engineers civil works project. Overall, the overlays were performing well with only minor shrinkage and reflective cracks observed. The majority of the repairs were made to combat continuing damage due to freezing and thawing of water-saturated nonair-entrained. Other overlays were employed to restore and protect concretes damaged by erosion and steel corrosion. A summary of these repairs is listed in Table 4 by type of damage being repaired. A summary of individual repairs is listed in Appendix A by type of overlay. Note that overlay unit costs do not include removal and patching costs. The repair unit costs in Appendix A include all costs.

The overlays that were employed to repair concrete damaged by freezing and thawing are typically performing *good to excellent*. Based on visits by the author to Mississippi River Lock Numbers 17, 18, 20, 21 and 22, Dashields Locks, Mahoning Creek Dam, and others, it appears that transverse cracks and cracks at corners of blockouts in the original concretes also exist in the overlays. Cracks in conventional and low-slump concretes appeared wider and longer than those for polymer-modified concretes and concretes containing fibers. The cracks in the concrete containing fibers (Mississippi River Lock Number 21) were smallest. Overall, the most cost effective overlay was one in which polypropylene fibers were added to the concrete mixture.

Concretes containing steel fibers have been used in the past to overlay bridge decks and warehouse floors. In 1990, the Federal Highway Administration (FHWA) sponsored a demonstration project in which a 5-mile (8.05 km) section of Interstate 10 in Louisiana was overlaid with a 4-in. (102-mm) thick concrete containing steel fibers (Anonymous 1992). The mixture proportions included 658 lb (298.5 kg) of cement and 85 lb (38.6 kg) of fibers per cubic yard. A follow-up demonstration is underway in which a 5.5-mile (8.85-km) section of I-10 is being overlaid with 3- to 5-in. (127-mm) thick concrete containing steel fibers. The overlaid pavements at both projects were continuously reinforced concrete. New design criteria may be forthcoming from the Interstate-10 work.

A polymer-modified concrete used to overlay the Mahoning Creek Dam Bridge Deck where damage due to freezing and thawing had occurred was performing good. The depth of the overlay was 1-1/2 in. (38 mm). The

overlay cost was the same as that for the polymer-modified concrete used on Tygart Dam roadway.

The concrete pavements of roadways on the tops of three dams had damage due to freezing and thawing. Two of the roadways were overlaid with unbonded conventional concretes and the third with a bonded polymer-modified concrete. The unbonded overlays were performing excellently and the polymer-modified overlay, good. These roadways are for maintenance of the dam only and have virtually no vehicular traffic. The depths of polymer-modified and the unbonded concretes were 1-1/2 in. (38 mm) and 4 in. (102 mm), respectively. The average cost for the unbonded overlay was significantly greater than for the polymer-modified.

One problem associated with the placement of polymer-modified concretes is assuring a timely application of the required wet curing after finishing. As polymer-modified concretes are more likely to develop shrinkage cracks than conventional and low-slump concretes when curing is delayed, there is a tendency for the placement personnel to rush the start of the wet curing. At two lock wall projects where polymer-modified concrete overlays were placed, the concrete was prematurely covered with burlap and polyethylene. This resulted in a wavy surface finish for these placements.

The cracks in the unbonded overlays at Dashields Locks and Mahoning Creek Dam were typically hairline and less frequent in occurrence than those observed in bonded overlay concretes. The use of dummy (control) joints reduced the number of cracks observed. The costs for the unbonded overlays were typically higher than those for bonded overlays.

The use of higher-strength concretes and abrasion-resistant aggregates resulted in good to excellent performances for overlays against abrasion-erosion damage. The most effective action to ensure good repair performance was to prevent the materials doing the abrading from returning after the repair. For stilling basins, prevention included sloping the upstream face of the end sill, raising the top elevation of the end sill, constructing a debris trap downstream of stilling basin, and paving the channel slopes along and downstream of the stilling basin. One project where poor performance occurred was overlaid with a conventional 3,000-psi (20.7-MPa) reinforced concrete and continued to have aggregate reentering the stilling basin after the repair. Remedial actions used to repair the overlay included patching the eroded areas with a 5,000-psi (34.5-MPa) concrete and eliminating the aggregate entry by securing the riprap along the stilling basin slopes with grout.

Steel-fiber-reinforced concrete is not recommended for use as an abrasion-erosion resistant overlay material. At one project, a steel-fiber-reinforced concrete overlay was placed in a stilling basin to repair abrasion-erosion damaged concrete. After about 10 years of service, the stilling basin was again in need of repair. The poor performance of the fiber-reinforced concrete was also seen in abrasion test results from an evaluation of concrete mixtures for the stilling basin repair (Holland 1983). It was concluded that

the fiber-reinforced concrete performed no better that the original concrete in resisting abrasion erosion (Holland and Gutschow 1987).

Silica-fume and fly-ash concretes were used to overlay Los Angeles River channel pavements where abrasion damage had occurred. The silica-fume concrete was nearly twice the strength of the fly-ash concrete and more costly. Both overlays were performing excellently.

The fly-ash concrete overlay used to repair cavitation erosion damage has not been subjected to cavitation producing flow since the repair, and therefore, its performance can not be evaluated. Other repairs to cavitation damaged concrete were previously documented in Technical Report REMR-CS-16 (Campbell and Bean 1988). A testing program funded by the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program is presently ongoing to evaluate materials and techniques for repair of cavitation damaged concrete. A report of the results is in preparation.

Overlays were used to repair the decks of three highway bridges where steel corrosion damage had occurred. Two of the overlays were low-slump concretes having 1-3/4-in. (45-mm) and 2-1/4-in. (57-mm) thicknesses, respectively. Both low-slump overlays are performing good. The third overlay was a 1-1/2-in. (38-mm) thick polymer-modified concrete and is performing fair. The cost for overlays were approximately the same.

A study for the U.S. Department of Transportation (Abdulshafi, Kvammen, and Kaloush 1990) regarding the premature failure of latex-modified concrete bridge deck overlays in Ohio showed that 28 percent of the repairs were performing less than fair. The general findings were that most performance problems were not the result of inadequacies of the overlay material but were the result of insufficient prerepair testing to determine the extent of chloride-ion penetration. This resulted in chloride-ion contaminated concrete underlying the overlay and subsequent performance problems. The newest technology for chloride-ion detection is a chloride-ion electrode probe developed under the Strategic Highway Research Program (SHRP). The use of the probe significantly reduces analysis time and is expected to be included as part of the standard practice for chloride-ion identification.

A steel-wire-reinforced, polymer-modified mortar was used to overlay chloride-ion damaged concrete balconies. The depth of the overlay was 7/8 in. (22 mm). The cost for the overlay was higher than for low-slump and polymer concretes used to repair chloride-ion damaged concrete in bridge decks. The repair has been in service for less than a year with no signs of distress. Long-term performance data are needed to complete an evaluation for the mortar's use as a overlay material. Additional study is needed to evaluate its applicability for service under vehicular traffic conditions.

A laminated ferrocement, a boat building material, has been used to resist corrosion in boat hulls and decks for over 20 years and is being proposed as a overlay material for bridge decks (Iorns 1992). The general application procedure consists of spraying a latex bond coat, spraying two layers (each

approximately 1/8 in. (3 mm) thick) of latex-modified, and pressing a layer of steel mesh into the mortar. A third layer of mortar and mesh are applied followed by a fourth layer of mortar. The outer 1/8-in. (3-mm) thick layer contains no steel and serves as cover for the steel mesh. The total thickness of the overlay is approximately 1/2 in. (13 mm). The water-cement ratio is specified to be below 0.4 and the cement-sand ratio above 0.5. The most cost-effective steel mesh proved to be an expanded metal plaster lath at 3.4 lb/yd^2 (1.69 kg/m^2). Costs and long-term performance data are needed to evaluate its applicability as an overlay material for bridges and other types of structures.

Two highway roadways were overlaid with a 7-in. (178-mm) thick, unbonded, reinforced concrete (Simonsen and Price 1989). One concrete was a conventional mixture and the other, a fly-ash. Both repairs were performing good under high traffic volumes.

The use of polymer concrete to overlay bridge decks is becoming more common. The polymer concretes are generally placed as a single-layer placement having a minimum thickness of 3/4 in. (19 mm) or a multilayer placement consisting of approximately 1/8-in. (3 mm) thick layers having a total minimum thickness of 3/8 in. (10 mm). The binder for the polymer concrete is usually an epoxy, polyester, or methacrylate and the aggregate a silica or basalt (Tarricone 1992). Both the aggregate and bond surface are required to be dry at time of placement. The polymer concrete is highly impermeable and can reach strengths of around 1,500 psi (10.3 MPa) in 3 hr and 5,000 psi (34.5 MPa) in 24 hr. Virginia, California, and New York are the leading users of polymer concrete, with Virginia placing 8 to 10 bridge deck overlays per year. The service life of a polymer overlay is expected to be 10 to 20 years. The cost is generally higher than those of low-slump and latex-modified concretes.

Table 4
Summary of Overlay Repairs

Damage	Structures		Type Overlay	Range					Performance
	Element	No.		Completion Date	Compressive Strength psi (MPa)	Area yd^2 (m^2)	Depth in. (mm)	Cost $\$/yd^2$ ($\$/m^2$)	
Freezing and Thawing	Lock Wall	3	Conventional	1982-1989	3,000-4,000 (20.7-27.6)	2,900-4,330 (2,430-3,620)	4-10 (102-254)	130-135 (155-161)	Good
		3	Low-Slump	1987-1991	4,000 (27.6)	2,340-3,510 (1,960-2,940)	4 (102)	44-81 (53-97)	Good - Excellent
		2	Polymer-Modified	1989-1991	-	2,540 (2,130)	1 1/2 (38)	8-50 (10-60)	Excellent
		3	Fiber-Reinforced	1987	-	880-1,060 (736-886)	2 (51)	24-37 (29-44)	Excellent
		2	Unbonded	1989	3,000-4,000 (20.7-27.6)	4,460 (3,720)	12 (305)	170 (203)	Excellent
	Bridge Deck	1	Polymer-Modified	1987	-	780 (652)	1 1/2 (38)	40 (48)	Good
	Roadway	1	Polymer-Modified	1986	-	1,220 (1,020)	1 1/2 (38)	40 (48)	Good
		2	Unbonded	1987	3,000 (20.7)	205-795 (171-665)	4 (102)	29-180 (35-215)	Excellent
Abrasion	Stilling Basin Tunnel	2	Conventional	1983-1984	3,000-5,000 (20.7-34.5)	110-2,900 (92-2,420)	2-12 (51-305)	-	Poor-Good
		1	Abrasive-Resistant Aggregate	1975	6,000 (41.4)	2,100 (1,760)	10 (254)	51 (61)	Good
		1	Steel-Fiber-Reinforced	1973-1974	6,000 (41.4)	3,440 (2,880)	12 (305)	143 (171)	Failed
		1	Silica-Fume	1983	12,500 (86.2)	3,440 (2,880)	12 (305)	90 (108)	Good

(Continued)

Table 4 (Concluded)

Damage	Structures: Element	Structures: No.	Type Overlay	Range: Completion Date	Range: Compressive Strength psi (MPa)	Range: Area yd^2 (m^2)	Range: Depth in. (mm)	Range: Cost $\$/yd^2$ ($\$/m^2$)	Performance
Abrasion	Channel	1	Fly-Ash	1988	5,000 (34.5)	733,000 (613,000)	6 (152)	21 (25)	Excellent
		1	Silica-fume	1985	8,000-10,500 (55.2-72.4)	160,000 (134,000)	6 (152)	26 (31)	Excellent
	Tunnel	1	Fly-Ash	1990	7,000 (48.3)	1,650 (1,380)	12 (305)	101 (121)	Excellent
Cavitation	Tunnel	1	Silica-Fume	1991	10,000 (69.0)	916 (766)	9 (229)	715 (855)	-
Steel Corrosion	Bridge Deck	2	Low-Slump	1982-1987	4,000 (27.6)	1,820-1,900 (1,520-1,590)	1 3/4-2 1/4 (44-57)	33 (39)	Good
		1	Polymer-Modified	1985	-	23,800 (19,900)	1 1/2 (38)	34 (41)	Fair
	Balcony	1	Reinforced, Polymer-Modified	1992	4,000 (27.6)	270 (226)	7/8 (22)	105 (126)	Excellent
Vehicular	Roadway	2	Unbonded	1984	5,900 (40.7)	8,300 (6,940)	7 (178)	-	Good

10 Conclusions and Recommendations

Good to excellent performance can typically be expected from both bonded and unbonded overlays of properly air-entrained concrete made using sound aggregate and protected from exposure until adequately mature for concretes of which the surface damage was due to a lack of resistance to freezing and thawing while critically saturated. However, to provide a surface with minimal cracking, the use of either extensive dummy joints or unbonded overlays was required.

Shrinkage and reflective cracks are common in bonded overlays. Those that develop in polymer-modified concretes and concretes containing polypropylene fibers are more likely to be smaller in width and length than those of similar cracks in conventional and low-slump concretes. Cracks in concretes containing fibers are generally smallest. Cracks that exist at blockouts and other changes of geometry in the original concrete will likely be reflected in the bonded overlay. The location of these cracks can be controlled via employment of dummy joints.

The standard practice for bonded overlays is to remove the deteriorated concrete, to create a near uniform profile by patching surface depressions that are greater than 2 in. (51 mm) deep with conventional concrete, to prepare the surface for placement, and to place a 3,000- to 4,000-psi (20.7- to 27.6-MPa) concrete having a minimum depth of 2 in. (51 mm) for conventional, low-slump, or polypropylene-fiber concretes and 1-1/2 in. (38 mm) for polymer-modified concretes. A diagonally placed reinforcing bar is typically included at corners of blockouts to limit cracking at these corners.

Higher-strength concretes are often required when service environment includes vehicular traffic or erosion. For overlays that are subjected to vehicular traffic, the concrete design depth and strength are based on the average daily traffic.

Surface preparation for bonded overlays typically includes preparing the surface by shotblasting, sandblasting, or mechanical abrading; removing dust and other loosely adhered materials by waterblasting or air blasting; and applying a bond coat just prior to the placement of the overlay. The bond coat is most often the mortar portion of the overlay concrete, although the use

of epoxy bonding agents to bond cementitious based concretes is not uncommon.

Unbonded overlays typically have less frequent and smaller cracks than bonded overlays. The unbonded overlays sometimes include dummy joints to control crack locations. These joints are filled with sealant to block access of water and chloride ions to the concrete interior. The standard practice for unbonded overlays is to create a uniform profile by patching surface depressions with conventional concrete, to install a bond breaker, and to place a 3,000- to 4,000-psi (20.7- to 27.6-MPa) conventionally reinforced concrete having a minimum depth of 4 in. (102 mm). A fly-ash concrete is sometimes used in lieu of conventional concrete when overlay is 7 in. (178 mm) thick and greater. The bond breaker used at reported projects included layers of 15-lb (6.80-kg) asphalt-saturated felt, asphalt-modified urethane membrane, and a sand-asphalt mixture. Surface preparations for unbonded overlays are typically not required or minimal.

For unbonded overlaying of roadways, areas of roadway that would provide inadequate support are repaired prior to overlaying. A study funded by the Federal Highway Administration (Voigt, Carpenter, and Darter 1989) recommends that a minimum 1-in. (25-mm) thick hot-mix asphalt concrete used as a bond breaker (polyethylene sheeting is not recommended due to its low friction factor). The study also concludes that the joint spacing for the overlays should be limited to 15 ft (4.6 m) or less; however, if longer joint spacing is used then heavy reinforcement must be included to keep the cracks tight. Dowels are included at joints for load transfer. Joints in overlay should be located a minimum of 3 ft (0.914 m) from existing joints and working cracks in roadway (Voigt, Carpenter, and Darter 1989).

The standard practice for overlaying bridge decks is to remove deteriorated concretes, to prepare surface for placement, and to place a 3,000- to 4,000-psi (20.7- to 27.6-MPa) bonded concrete having a minimum depth of 2 in. (51 mm) for low-slump concrete and 1-1/2 in. (38 mm) for polymer-modified concrete. The placement of polymer concrete having a minimum depth of 3/4 in. (19 mm) is an accepted practice where an impermeable, high-strength, fast-curing material is required.

The chloride-ion electrode probe developed under SHRP should be used to facilitate the detection of chloride-ion contamination. In the past, prerepair testing to determine the extent of chloride-ion penetration was sometimes inadequate. This resulted in chloride-ion contaminated concrete being overlaid and subsequent damage to the overlay due to continued corrosion in the underlying concrete.

The standard practice for repairing abrasion-erosion damaged concrete includes taking remedial actions that will prevent the materials doing the abrading from returning after the repair and overlaying with a more abrasion-resistant concrete. The use of 6,000-psi (41.4-MPa) or greater strength concretes and abrasion-resistant aggregates will enhance the overlay's performance against abrasion damage. Silica-fume concrete overlays perform

better than conventional concrete overlays. The use of steel-fiber-reinforced concrete is not recommended.

The expected service life for concrete overlays differs among types of structures. A service life of 10 to 20 years is expected for bridge decks, 15 years or more for highways, and 20 years or more for mass-concrete structures. For abrasion damaged structures, service life is expected to be 20 years or more if the material causing the abrasion is prevented from returning. In general, the more flexible a structure is, the shorter the expected service life for the overlay.

The extension of service life resulting from using one type of overlay in lieu of another was not evident in the case histories examined, with the exception of where fiber-reinforced concrete was used to resist abrasion erosion.

Based on the results of this study, it would benefit future overlay work at Corps of Engineers projects if guidance were provided for (a) determining extent of chloride-ion penetration and subsequent removal, (b) selecting dummy joint locations and materials, (c) selecting and installing a bond breaker, (d) identifying when wet curing of polymer-modified concrete can begin, and (e) placing overlay concretes containing steel fibers for repair of nonerosion damaged concrete based on results of FHWA demonstration projects in Louisiana. The guidance of determining extent of chloride-ion penetration should include SHRP's newly developed chloride-ion electrode probe. The guidance for removal should include chloride-ion extraction methods.

It would be of benefit to the Corps of Engineers if acceptance criteria for bond breakers were developed. It would also be of benefit if steel-reinforced polymer-modified concretes were evaluated as an overlay material for bridge decks and marine structures.

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Appendix A
Summary of Individual Repairs by Type of Overlay

Project	Reason for Repair	Type Overlay	Repair Date	Overlay				Repair Unit Cost $/yd² ($/m²)	Performance
				Depth in. (mm)	Area yd² (m²)	Compressive Strength psi (MPa)	Unit Cost $/yd² ($/m²)		
Black Rock Lock	Deterioration due to Freezing and Thawing	Welded-wire-reinforced, conventional concrete	1982, 1988	4 (102)	-	4,000 (27.6)	-	-	Reflective cracking
Clearwater Dam Stilling Basin	Abrasion erosion	Conventional concrete	1984	2 (51)	110 (92)	5,000 (34.5)	-	500 (598)	Good
DeQueen Dam Stilling Basin	Abrasion erosion	Reinforced, conventional concrete	1983	12 (305)	2,900 (2,420)	3,000 (20.7)	-	-	Abrasion erosion along upstream construction joint and around upstream baffle blocks in which the most extensive damage was 10-in. (254-mm) deep; large pattern cracks
Emsworth Locks	Deterioration due to Freezing and Thawing	Reinforced, conventional concrete	1982-1986	12 (305)	2,900 (2,420)	3,000 (20.7)	135 (161)	-	Traverse cracks, cracking at gate anchors, and two monoliths with heavy pattern cracking

Project	Reason for Repair	Type Overlay	Repair Date	Overlay: Depth in. (mm)	Overlay: Area yd^2 (m^2)	Overlay: Compressive Strength psi (MPa)	Overlay: Unit Cost $\$/yd^2$ ($\$/m^2$)	Repair Unit Cost $\$/yd^2$ ($\$/m^2$)	Performance
Montgomery Locks	Deterioration due to Freezing and Thawing	Reinforced, conventional concrete	1985-1989	12 (305)	4,320 (3,620)	3,000 (20.7)	130 (155)	-	Minor traverse cracking; more extensive cracking in gate monoliths and at blockouts; several sections developed pattern cracking over an entire slab
Tuttle Creek Dam Stilling Basin	Abrasion erosion	Reinforced, conventional concrete containing abrasion-resistant aggregate	1975	10 (254)	2,100 (1,760)	6,000 (41.4)	51 (61)	91 (109)	Good condition with minor cracking
Mississippi River Lock Number 17	Deterioration due to Freezing and Thawing	Low-slump concrete	1989-1990	4 (102)	3,510 (2,940)	4,000 (27.6)	44 (53)	159 (190)	Minor cracking at corners of blockouts
Mississippi River Lock Number 18	Deterioration due to Freezing and Thawing	Low-slump concrete	1991	4 (102)	-	4,000 (27.6)	-	-	Minor cracking at corners of blockouts

Project	Reason for Repair	Type Overlay	Repair Date	Overlay				Repair Unit Cost \$/yd² (\$/m²)	Performance
				Depth in. (mm)	Area yd² (m²)	Compressive Strength psi (MPa)	Unit Cost \$/yd² (\$/m²)		
Mississippi River Lock Number 20	Deterioration due to Freezing and Thawing	Low-slump concrete	1987	4 (102)	2,340 (2,000)	4,000 (27.6)	81 (97)	166 (199)	Minor cracking at corners of blockouts
Red Rock Dam Spillway Bridge	Steel Corrosion	Low-slump concrete	1982	1-3/4 (44)	1,820 (1,520)	-	33 (39)	61 (73)	Minor shrinkage cracks
Tuttle Creek Dam Spillway Bridge	Steel Corrosion	Low-slump concrete	1987	2-1/4 (57)	1,900 (1,590)	4,000 (27.6)	-	-	Good
Los Angeles River Main Channel	Abrasion erosion	Fly-ash concrete	1986-1988	6 (152)	733,000 (613,000)	5,000 (34.5)	-	21 (25)	Excellent
Mojave River Forks Dam Outlet Tunnel	Abrasion erosion	Fly-ash concrete	1990	12 (305)	1,650 (1,380)	7,000 (48.3)	101 (121)	302 (361)	Excellent after 1 yr service
Kinzua Dam Stilling Basin	Abrasion erosion	Silica-fume concrete	1983	12 (305)	3,440 (2,880)	12,500 (86.2)	90 (108)	394 (471)	Good with 2- to 8-in. (51- to 203-mm) depths of erosion observed along nearly the entire upstream floor
Los Angeles River Low-Flow Channel	Abrasion erosion	Silica-fume concrete	1983-1985	6 (152)	160,000 (134,000)	9,000 (62.1)	26 (31)	28 (33)	Excellent

Project	Reason for Repair	Type Overlay	Repair Date	Overlay				Repair Unit Cost $/yd^2 ($/m^2)	Performance
				Depth in. (mm)	Area yd^2 (m^2)	Compressive Strength psi (MPa)	Unit Cost $/yd^2 ($/m^2)		
Lowell Creek Dam Diversion Tunnel	Cavitation erosion	Silica-fume concrete	1991	9 (229)	916 (766)	10.000 (69.0)	715 (855)	770 (921)	Possible minor erosion at center of invert, however, no excessive discharge flows were reported
Apartment Balconies: Guelph, Ontario, Canada	Steel Corrosion	Reinforced, polymer-modified mortar	1992	7/8 (22)	-	4,000 (27.6)	105 (126)	105 (126)	-
Mahoning Creek Dam Spillway Bridge	Deterioration due to Freezing and Thawing	Polymer-modified concrete	1987	1-1/2 (38)	780 (652)	-	40 (48)	100 (120)	Numerous fine cracks
Mississippi River Lock Number 3	Deterioration due to Freezing and Thawing	Latex-modified concrete	1990-1991	1-1/2 (38)	-	-	8 (10)	-	Minor cracking along half of the repair edges and within about 10 percent of areas repaired
Mississippi River Lock Number 22	Deterioration due to Freezing and Thawing	Latex-modified concrete	1989	1-1/2 (38)	2,540 (2,130)	4,000(27.6)	50 (60)	89 (106)	Minor cracking at corners of blockouts

Project	Reason for Repair	Type Overlay	Repair Date	Overlay				Repair Unit Cost $/yd^2 ($/m^2)	Performance
				Depth in. (mm)	Area yd^2 (m^2)	Compressive Strength psi (MPa)	Unit Cost $/yd^2 ($/m^2)		
Reedy Point Bridge	Steel Corrosion	Latex-modified concrete	1985	1-1/2 (38)	23,800 (19,900)	4,000 (27.6)	34 (41)	41 (49)	Numerous areas of hairline map cracking accompanied by hollow sounding concrete; some traverse cracking near joints; medium map cracking (one span)
Tygart Dam Roadway	Deterioration due to Freezing and Thawing	Polymer-modified concrete	1986	1-1/2 (38)	1,220 (1,020)	-	40 (48)	56 (67)	Reedy Point Bridge
W. G. Huxtable Pumping Plant	Infiltration of water	Acrylic-resin modified concrete	1986	1/4 (6)	2,110 (1,760)	-	-	96 (115)	Small amounts of leakage
LaGrange Lock	Deterioration due to Freezing and Thawing	Polypropylene-fiber-reinforced concrete	1987	2 (51)	1,060 (886)	-	37 (44)	92 (110)	Minor cracking at corners of blockouts
Mississippi River Lock Number 21	Deterioration due to Freezing and Thawing	Polypropylene-fiber-reinforced concrete	1987	2 (51)	882 (737)	-	24 (29)	60 (72)	Minor cracking at corners of blockouts

Project	Reason for Repair	Type Overlay	Repair Date	Overlay				Repair Unit Cost \$/yd² (\$/m²)	Performance
				Depth in. (mm)	Area yd² (m²)	Compressive Strength psi (MPa)	Unit Cost \$/yd² (\$/m²)		
Peoria Lock	Deterioration due to Freezing and Thawing	Polypropylene-fiber-reinforced concrete	1987	2 (51)	880 (736)	-	32 (38)	152 (182)	Minor cracking at corners of blockouts
Kinzua Dam Stilling Basin	Abrasion erosion	Steel-fiber-reinforced concrete	1973-1974	12 (305)	3.440 (2,880)	6,000 (41.4)	143 (171)	499 (597)	Failed after 10 years service; 36-in. (914-mm) deep holes in stilling basin floor
Dashields Locks	Deterioration due to Freezing and Thawing	Unbonded, reinforced, fly-ash concrete	1989	12 (305)	4,460 (3,720)	4,000 (27.6)	170 (203)	170 (203)	Excellent condition with a reduction in occurrence of cracking from areas previously placed without bond breakers
Interstate 96 at Portland, Michigan	Vehicular damage	Unbonded, reinforced, conventional concrete	1984	7 (178)	8,300 (6,940)	5,970 (41.2)	-	-	Satisfactory, Cracking more frequently located near or at cracks or joints in the original pavement beneath

Project	Reason for Repair	Type Overlay	Repair Date	Overlay				Repair Unit Cost $/yd^2 (\$/m^2)	Performance
				Depth in. (mm)	Area yd^2 (m^2)	Compressive Strength psi (MPa)	Unit Cost $/yd^2 (\$/m^2)		
Loyalhanna Dam Roadway	Deterioration due to Freezing and Thawing	Unbonded, conventional concrete	1987	4 (102)	205 (171)	3,000 (20.7)	180 (215)	-	Only occasional fine cracks
Mahoning Creek Dam Roadway	Deterioration due to Freezing and Thawing	Unbonded, conventional concrete	1987	4 (102)	795 (665)	3,000 (20.7)	29 (35)	-	Only occasional fine cracks
Montgomery Locks	Deterioration due to Freezing and Thawing	Unbonded, reinforced, conventional concrete	1989	12 (305)	-	3,000 (20.7)	-	-	cracking in the unbonded concrete resurfacing significantly less than cracking in bonded resurfacing
U.S. Highway 23 near Dundee, Michigan	Vehicular damage	Unbonded, reinforced, fly-ash concrete	1984	7 (178)	8,300 (6,940)	5,820 (40.1)	-	-	Satisfactory, cracking more frequently located near or at cracks or joints in the original pavement beneath

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13. ABSTRACT *(Maximum 200 words)*

This study documents the current practices for overlaying horizontal concrete surfaces as a first phase in the development of performance criteria for concrete overlays. The case histories presented were typically for overlays completed within the last 10 years and located at Corps of Engineers civil works projects. Overlays documented included bonded conventional, low-slump, fly-ash, silica-fume, polymer-modified, and fiber-reinforced concretes. Unbonded overlays were also documented. Although the information obtained for each case history varied and was sometimes limited, an attempt was made to provide the following basic information for each repair: (a) project description, (b) cause and extent of damage, (c) description of repair materials and procedures, (d) cost, and (e) performance of repair.

The extension of service life resulting from using one type of overlay in lieu of another was not evident in the case histories examined, with the exception of where fiber-reinforced concrete was used to resist abrasion erosion (repair failed).

Shrinkage and reflective cracks that develop in polymer-modified concrete overlays and overlays containing polypropylene fibers are more likely to be smaller in width and length than those of similar cracks in conventional

(Continued)

14. SUBJECT TERMS

Bonded overlay	Fiber-reinforced concrete
Concrete overlay	Fly-ash concrete
Conventional concrete	Low-slump concrete

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15. NUMBER OF PAGES
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16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED	

ABSTRACT (Concluded)

and low-slump concrete overlays. Cracks in concretes containing fibers are generally smallest. Cracks that exist at corners of blockouts and other changes of geometry in the original concrete will likely be reflected in the bonded overlay. The location of these cracks can be controlled via employment of dummy joints. Unbonded overlays typically have less frequent and smaller cracks than bonded overlays.

It was recommended that guidance be provided for (a) determining extent of chloride-ion penetration and subsequent removal, (b) selecting dummy joint locations and materials, (c) selecting and installing a bond breaker, (d) identifying when wet curing of polymer-modified concrete can begin, and (e) placing overlay concretes containing steel fibers for repair of nonerosion damaged concrete. It was also recommended that acceptance criteria for bond breakers be developed and that steel-reinforced polymer-modified concretes be evaluated as overlay materials for bridge decks and marine structures.

SUBJECT TERMS (Concluded)

Polymer-modified concrete
Silica-fume concrete
Thin overlay
Unbonded overlay